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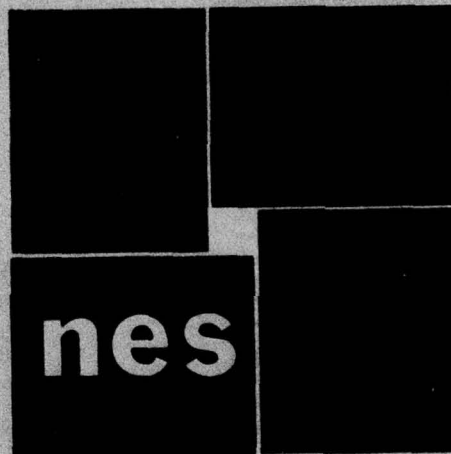
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DESIGN OF WAVE TANKS



FINAL REPORT

Contract No. DA 22-079 CIVENG 62-46

NATIONAL ENGINEERING SCIENCE CO.

Prepared For:

**U.S. Army Engineer
Waterways Experiment Station
Vicksburg, Mississippi**

AD-A054080

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DESIGN OF WAVE TANKS

Contract No. DA 22-079 CIVENG 62-46

FINAL REPORT

April 1962

by

NESCO STAFF

NATIONAL ENGINEERING SCIENCE CO.
711 South Fair Oaks Avenue
Pasadena, California

Sponsored by

U. S. Army Engineer
Waterways Experiment Station
Vicksburg, Mississippi

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Prepared by:

NESCO Staff

Approved:

Lars Skjelbreia

Lars Skjelbreia, Ph.D.
Vice President for Engineering

L. B. McCammon

L. B. McCammon, Ph.D.
Associate Director of Engineering

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NOTATION

T	: Wave period, $k = \frac{2\pi}{T}$
d	: Depth and depth at the paddle
d_t	: Depth in the testing area
L	: Wave length, $m = \frac{2\pi}{L}$ (at the wave paddle)
ℓ	: Length of the wave paddle and width of the wave tank at the wave paddle section
ℓ_t	: Width of the wave tank in the testing area
H	: Actual wave height generated by the paddle in the wave paddle section
H_{th}	: Theoretical wave height generated by the paddle
η	: Efficiency of the paddle, $H = H_{th} \times \eta$
G	: Group velocity at the wave paddle
G_t	: Group velocity in the testing area
S	: Scale
α	: Slope of wave absorber
γ	: Wave steepness $\frac{H}{L}$
γ_o	: Wave steepness in deep water
R	: Reflection coefficient (Ratio of reflected wave height to incident wave height)
D	: Damping coefficient due to wave filter
λ	: Friction coefficient due to wave filter
ε	: Void coefficient of wave filter ($\varepsilon \neq 1$)
β_1	: Reflection coefficient of the structure under study in the wave tank
β_2	: Reflection coefficient of the wave paddle

$$A = D^2 \beta_1 \beta_2$$

$$m = \frac{2\pi}{L}$$

$2e$: Stroke at the free surface

$2 \xi(z)$: Stroke as a function of the distance z on a vertical

$$K = \frac{H_{th}}{2e} \quad \text{in the case of a piston type paddle}$$

$$K' = \frac{H_{th}}{2e} \quad \text{in the case of a hinged type paddle}$$

θ : $\text{Tang } \theta = \frac{e}{d}$: stroke in terms of the maximum angle of the wave paddle with the vertical

N : Number of wave filter elements

x : Damping length due to wave filter

Γ : Damping coefficient due to one wave filter element

P_1 : Total force on the paddle due to wave motion = $X_1 \cos kt$

P_w : Force per unit of area on the paddle due to wave motion

P_2 : Total force on the paddle due to water inertia = $X_2 \cos kt$

P_i : Force per unit of area on the paddle due to water inertia

m_n : Defined by $k^2 = m_n^2 \text{ tang } m_n d$

W : Wave power in H. P.

ρ : Fluid density

g : Acceleration due to gravity

n : An integer

z : Distance from the bottom

C : Propagation velocity

SUMMARY

Introduction

This report deals with the design of two wave tanks for studying breakwater stability. A wave basin (W. B.) is designed for three-dimensional studies. A wave flume (W. F.) is designed for two-dimensional studies at a larger scale.

Section One - Determination of the Main Dimensions

I. The Design Wave Criteria and Scales

Normal operating scales are $1/25$ in the W. F. and $1/50$ in the W. B. The range of periods is 0.7 - 4.2 sec. in the W. F. and 0.5 - 3.0 in the W. B.

II. Test Areas

Normal operating depths are 4 ft. in the W. F. and 2 ft. in the W. B. but could be greater (4.5 ft and 3 ft. respectively). The depth of the W. F. is 7.5 ft. and the depth of the W. B. is 5.5 ft.

A distance is provided for measuring the wave height in the W. F. A lateral measurement W. F. is designed within the W. B.

The width of the W. F. is 4 ft. to avoid transverse resonance. The width of the W. B. is 50 ft. and vertical guides limit transverse resonance. The W. B. is L-shaped for studying end breakwater at an angle. The end breakwater must be constructed out of the diffraction area.

III. Wave Absorbers

Areas for wave absorbers exist at both ends of the tanks. The design consists of rocky beaches of gentle slope near the M. W. L. and are ended by a vertical wall in deep water for saving space.

IV. Wave Filter

Wave filters permit good boundary conditions at the side of the wave paddle. The required damping coefficient is 0.5 for a vertical breakwater, 0.8 for a rockfill breakwater, and 1.0 for a beach.

The wave filter designs are based on removable plastic material suspended vertically from horizontal bars. The number of filters required must be determined by trial and error.

Section Two - Wave Paddles

I. A Review of Wave Paddle Theories

A review of various wave paddle theories is given, including linear theories and non-linear theories, with sinusoidal and non-sinusoidal motion of the paddle.

II. Calculation of Wave Heights

The linear theory with sinusoidal motion is used to calculate the characteristics of the wave paddle. The design is based on wave paddles hinged at the bottom. An increase in depth of 2 ft. in the W. F. and 1.5 ft. in the W. B. is necessary. A convergent in the W. F. from 10 ft. to 4 ft. is also necessary for steep long waves.

The maximum stroke defined by $\tan \theta$ (angle of the wave paddle with the vertical) is 0.37 in the W. F. and 0.29 in the W. B.

It is possible to obtain the following wave characteristics

	Wave Height	Wave Period
W. F.	2 ft.	3.8 sec.
W. B.	1 ft.	2.7 sec.

in the testing area with a damping coefficient of 0.8 at the filter. Higher wave heights for smaller wave periods may also be obtained.

A general nomograph has been calculated for a hinged paddle and is presented in this report.

General nomographs for the W. F. and W. B., based on available theories, are also given and may be used to determine the wave characteristics in the testing areas as a function of the wave paddle adjustment.

III. Calculation of the Forces on the Wave Paddle and Power Requirements

The forces on the wave paddle for long waves are 17,000 lbs. in the W. B. and 10,000 in the W. F.

The forces on the wave paddle for short waves of limit steepness are 4,000 lbs. in the W. B. and 2,640 lbs. in the W. F. No available theories are valid for breaking waves. A safety coefficient is recommended for waves breaking off the paddle when too great a stroke is used with a small wave period.

The maximum wave power is 13.7 H. P. in the W. F. and 16 H. P. in the W. B. It is recommended that the motor power be 60 H. P. in the W. B. and also in the W. F.

IV. Generation of Irregular Waves

It is possible to produce irregular waves by changing the wave period continuously.

It will be possible, by a slight modification, to produce a wave train by periodically changing the stroke of the paddle.

INTRODUCTION

This report deals with the complete design of two wave tanks planned to be built at the Waterways Experiment Station, Vicksburg, Mississippi.

The primary purpose of these two wave tanks is the study to the stability of rockfill breakwaters in scale model. However, they can also be used for other experimental studies related to wave hydrodynamics and coastal engineering.

The first wave tank is designed to study the stability of a breakwater, including the breakwater head, attacked by waves at an angle. In this report this wave tank will be called a wave basin (W. B.) to differentiate it from the second wave tank.

The second wave tank will be called a wave flume (W. F.). It is designed to carry out two-dimensional studies of breakwater sections with a range of scale which is greater than the usual range. This should permit avoidance of scale effects without having too great a scale, which involves costly experiments. In this wave flume, the effect of a number of important parameters on breakwater stability can be carefully investigated; for example, the method of construction, effect of perviousness, etc.

The design of the two wave generators has been carried out with provisions for necessary modification for reproducing irregular waves.

In this report the choice of design criteria, the complete hydraulic design, structural design, mechanical design and constructions for constructing and operating the two wave tanks are given successively. A brief

summary is given of the elements of wave tank theories in order to justify the selection of the design dimensions. These general wave tank theories are also valid for other wave tanks. Recent and unpublished additions to these theories have been developed prior to or in relation to this project. Since a number of wave tank theories are common to these two wave tanks (and to any wave tank), it is more convenient to present the calculations for the hydraulic design for both tanks simultaneously, using the initials W. B. and W. F. to differentiate between the two tanks.

SECTION ONE

DETERMINATION OF THE MAIN DIMENSIONS

I THE DESIGN WAVE CRITERIA AND SCALES

The choice of wave criteria and wave tank dimensions depends upon the range of wave characteristics at sea and the range of possible scale to be used in the wave tanks.

I-1) Wave Characteristics at Sea

The wave period which is of interest in coastal engineering varies from $T = 5$ sec. for the Great Lakes to $T = 18$ or 20 sec. in the Pacific Ocean. A wave period of $T = 10$ sec. is common.

The maximum wave height encountered at sea may be as high as $H = 100$ feet. However, such a wave height will be reduced considerably on the continental shelf by friction and white cap effects. The maximum wave height is in fact very often limited by the depth. Rarely is a breakwater attacked by a wave of greater amplitude than $H = 50$ feet. A depth greater than 100 feet for the construction of a breakwater is rarely encountered.

I-2) Choice of Scales

I-2. 1) A scale too small results in scale effects and error. A scale too large can be costly. The optimum scale should be calculated by comparing the economics of the scale model study and that of the experiment. The safety factor and the saving increase with the scale. The cost of experiment increases with scale. Hence the difference presents an optimum which gives the best scale. Unfortunately such a preliminary investigation is impossible

and the scales have to be chosen by experience after a long (say 20 years) trial and error.

I-2. 2) The usual scales for studying breakwater stability are $1/50 - 1/40$ in W. F. and $1/70 - 1/50$ in W. B. Since the primary purpose of the two wave tanks (and particularly of W. F.) is to improve the actual data by getting rid of scale effects, it may be considered that the normal operating scales will be $1/25$ in W. F. and $1/50$ in W. B. These two scales will be the scale criteria for the design of the wave tanks.

I-2. 3) A larger range of scale may be considered for studying breakwater stability first, but also for carrying out other studies in which case the wave tanks become multiple purpose facilities. For example, some studies of wave agitation or littoral drift may be carried out in the wave basin after construction of bottom contours on the fixed slab of the basin. Wave agitation in a harbor is studied at a scale as small as $1/150$ without capillarity effect or without too much damping by viscous friction. Littoral drift could be studied at a scale as small as $1/300$ for the horizontal scale and $1/100$ for the vertical scale.

W. F. could also be used for studying the transport of sand by wave and current, and the perviousness of rock fill breakwater to long waves, of particular importance in seiche problems.

Hence, the possible use of the two wave tanks for other kinds of studies without additional cost will be kept in mind.

The desirable range of scale is presented below:

TABLE 1

	<u>Maximum</u>	<u>Minimum</u>	<u>Normal</u>
W. F.	$1/10$	$1/100$	$1/25$
W. B.	$1/25$	$1/150$	$1/50$

It should be emphasized that these figures are not precise but give only a general order of magnitude.

I-3) Range of Period

I-3. 1) The maximum range of period to be reproduced is obtained by considering the maximum scale with the longest wave period at sea on one hand, and the minimum scale and the shortest wave period at sea on the other hand.

The wave period at sea T_g and the wave period in the tanks T_t are related by the formula:

$$T_t = T_g \sqrt{S}$$

where S is the scale of the model. Hence, by combining the range of period, 5 sec. $< T_g < 18-20$ sec., with the range of desirable scale, the range of wave period for the tanks is determined and summarized in Table 2 below:

TABLE 2

	<u>Maximum</u>	<u>Minimum</u>	<u>Normal</u>
W. F.	20 $\sqrt{1/10}$ = 6.3 s.	5 $\sqrt{1/100}$ = 0.5 s.	10 $\sqrt{1/25}$ = 2.0 s.
W. B.	20 $\sqrt{1/25}$ = 4.0 s.	5 $\sqrt{1/150}$ = 0.41 s.	10 $\sqrt{1/50}$ = 1.4 s.

I-3. 2) The above range in wave periods is limited by the limits of the mechanical (or electrical*) variator inserted in the wave generator system. Common ranges of variation are 1 - 6, 1 - 8, 1 - 10.* Depending on the range of this variation, the following wave periods are reproduced:

*This part was written prior to the mechanical design of the wave generator.

TABLE 3

Mechanical Range		1 - 6	1 - 8	1 - 10
W. F.	T sec.	0.7 - 4.2	0.6 - 4.8	0.55 - 5.5
W. B.	T sec.	0.5 - 3.0	0.45 - 3.6	0.4 - 4.0

By combining the results of Table 2 and Table 3 it may be seen that a mechanical range of 1 - 6 is the minimum desirable range.

II TEST AREAS

II-1) Depths

II-1.1) The range of working water depth cannot be as flexible as the range of wave period. In fact, the scale of the model of a wave tank is often limited and determined by the maximum possible depth.

The maximum water depth for a breakwater of the deepest harbor is rarely greater than 100 feet. At the normal operating scale, the depth in the wave tank at the location of the breakwater will be:

$$\text{depth at sea} - \text{depth in wave tank} \times \text{scale}$$

which gives a depth of $100/25 = 4$ feet for W. F. and $100/50 = 2$ feet for W. B.

II-1.2) The worst condition for stability, particularly for a vertical breakwater, is encountered when the wave breaks in front of the structure. This condition requires the reproduction of the sea bottom in the wave tanks since breaking waves are due to relatively shallow water. For example, a wave of 40 feet in height will break at a depth of $d = \frac{H}{0.78} = \frac{40}{0.78} = 50$ feet, according to the solitary wave theory.

Therefore, the operating depth in front of the structure is reduced to $50/25 = 2$ feet for W. F. and $50/50 = 1$ foot for W. B.

II-1.3) The reproduction of refraction effects is of particular importance in the studies which will be carried out in W. B. The refraction effects become quite important when the water depth is less than one-third of the wave length ($d < L/3$). This condition requires reproduction of the bottom contours up to a depth of 100 feet for an eight second wave period and 200 feet

for an eleven second wave period. Hence, in most cases it would be unrealistic to adopt this criteria for the water depth which would lead one to chose wave tanks having too great a water depth or too small a scale.

It is possible to do a preliminary study to permit calculation of refraction. A wave refraction diagram gives the wave deformation between deep water and the corresponding depth where the wave paddle is located, from which one can determine the orientation of the bottom contours and breakwater in W. B. and the wave characteristics at the paddle.

This method is very reliable since it has to be used for relatively deep water where the linear theory is valid. The non-linear effects, which become important in shallow water and are even essential in the breaking zone, are reproduced in good similitude between the wave paddle and the structure.

II-1. 4) As a conclusion, the following depths may be considered as good operating depths:

		<u>Maximum</u>	<u>Normal</u>
W. F.	d feet	4.5	4
W. B.	d feet	3	2

Under normal conditions (depth of breakwater = 50 feet) the bottom contours may be reproduced between $d = 50$ feet and $d = 100$ feet or greater by using the maximum water depths. This is sufficient to reproduce in similitude the breaking phenomenon, if any, and the local wave refraction effects which are very important in three-dimensional studies.

II-1. 5) The maximum wave heights, given by $H_{\text{tank}} = H_{\text{sea}} \times \text{scale}$ are $50/25 = 2$ feet for W. F. and $50/50 = 1$ foot for W. B. The maximum rise of water against a vertical wall or wave run-up on a rubble mound slope may be as high as $\frac{3H}{2}$ above the M. W. L., i. e. 3 feet and 1.5 feet respectively, whence the maximum depths will be 7.5 feet for W. F. and 4.5 feet for W. B. As a conclusion, the above values are chosen.

II-1. 6) It would be convenient to have only one size of glass in the observation area in order to be able to interchange them if any of them are accidentally broken. A convenient size of glass is 4' x 5' which could be used vertically in W. F. and horizontally in W. B.

The glass five feet high in the W. F. will permit one to view the complete structure from the sea bottom to the top and also a large part of the bottom slope.

The glass four feet high in W. F. permits a relatively larger observation range.

The total number of glass panels will be $\frac{100}{4} = 25$ panels 4' x 5' for W. F. and $\frac{50 + 75}{4} = 25$ panels 5' x 4' for W. B.

II-2) Length of Test Areas and Measurements

II-2. 1) The length of test areas in the front of the structures must be calculated using the area over which the bottom contours have to be reproduced. The slope may be as small as 1/30 (or even less) and this would require a length of 60 feet in W. F. and 30 feet in W. B. However, in W. F. the slope can be exaggerated far from the breakwater.

The wave length is also a determining factor. The maximum wave lengths, for example, are $L = 56$ feet ($T = 5$ sec., $d = 4'$) for W. F. and $L = 30$ feet ($T = 3$ sec., $d = 3'$) for W. B. Hence the distance over which

the bottom contours need to be reproduced may be on the order of 30 feet in the W. F. and 30 feet in the W. B. because of the effects of three-dimensional refraction.

II-2. 2) A certain distance is required for measuring the incident wave height. This incident wave height will be given accurately in the W. F. by measuring the wave agitation at a node A and an antinode B of the partial clapotis in front of the structure on a horizontal bottom. $(H = \frac{A + B}{2})$.

For long waves in shallow water, the method proposed by Carry is more accurate. ⁽¹⁾ He takes the convective inertia forces at a second order of approximation into account.) These measurements could be made quickly and accurately by use of a scale against the glass. The distance between a node and an antinode is a quarter of a wave length. In fact, a slight transverse component due to the fact that the structure is not built exactly two-dimensionally may be a cause of error. Hence, three-fourths of a wave length for measurement is desirable for comparison between the wave agitation at two nodes and two antinodes.

The water surface elevations must not be influenced by bottom slope. Hence, $3/4 L = 40$ feet has to be retained in front of the last bottom contour; i. e. $40 \text{ ft.} + 30 \text{ ft.} = 70 \text{ ft.}$ A point gauge or electronic gauge in the W. F. may be used to obtain the wave height by averaging the value found on a single node and antinode on the two sides of the W. F.

II-2. 3) Measurement of the incident wave height in the W. B. cannot be made directly in front of the structure where there is a complex three-dimensional wave pattern. Hence a separate measurement wave flume

(M. W. F.) is necessary where the same incident wave is generated. At the end of this M. W. F. will be a very good wave absorber in order to determine the incident wave by a single measurement. Otherwise it will be necessary to measure nodes and antinodes. The width of the M. W. F. will be two feet so that it may be used also for carrying out other two-dimensional studies.

In summary, the test area in front of the structure will be 70 feet long in the wave flume and 30 feet in the wave basin.

II-3) Width of Test Areas

The W. F. must be as narrow as possible. Two-dimensional wave motion is a mathematical construction which never actually occurs, and the best way of approaching it is by narrowing the test area. The effect of boundary layers during wave motion due to later walls is on the order of a few millimeters. Any increase in width increases three-dimensional effects.

Of these three dimensional effects, the most important is that due to transverse resonance caused by any small dissymmetry in the W. F. or in the structure. It occurs when the width of the tank is equal to or near half a wave length or a multiple of half a wave length. The natural frequency of oscillation of a basin is obtained when its length is equal to $L/2$ or $n L/2$. Hence, transverse resonance may be avoided in the W. F. if the width is smaller than the smallest $L/2$, i. e. about 2.5 feet. Two and one-half feet are sufficient for a man to work in the flume and building of the scale model.

Another advantage of a small cross section is the economy realized on the material to handle the construction of the model. The study of stability of a rubble mound at random is of statistical nature and a great number of

stone rows is necessary. Roughly a 3" stone multiplied by 15 rows plus two rows influenced by wall effects gives 51".

Considering all advantages and disadvantages, a 4 foot wide W. F. is adopted.

II-3. 2) The wave basin, being three dimensional, will be subjected to transverse resonance. This can be solved by using lateral absorbers and vertical guides. A very wide W. B. is desirable to avoid wall effects. The width chosen is equal to two wave lengths, i. e. 50 feet, which permits a good reproduction of three-dimensional wave pattern far from the lateral walls.

II-3. 3) The W. B. is slightly deeper in the testing area in order that studies of local scouring due to wave motion at the toe of breakwaters can be carried out. An increase in depth of 6" has been considered for this purpose.

II-4) Diffraction Effects

II-4. 1) It is always difficult to conduct a three-dimensional test with lateral walls in a wave basin. The first solution exists in having a very wide flume with lateral wave absorbers and a very long wave paddle. Then the structure is built in front of the center of the wave paddle.

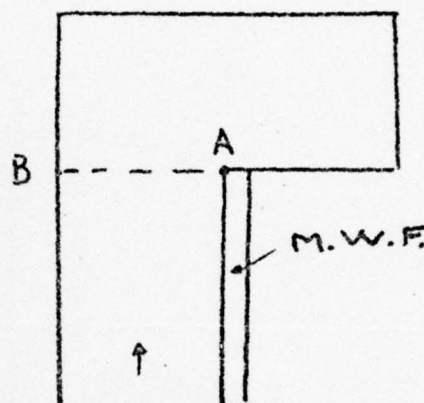
Since such a solution for studying the stability of a breakwater is not economical, it has been decided to operate between side walls. However, these side walls involve end effects, wave reflections, and transverse resonance. Vertical guides limit the transverse components. Moreover,

it has been decided to build an L-shaped wave basin. This L-shaped W. B. could be considered as an economical way of providing the same advantages as a larger W. B.

The enclosed sketches give the final dimensions and the various possible solutions for constructing an end breakwater.

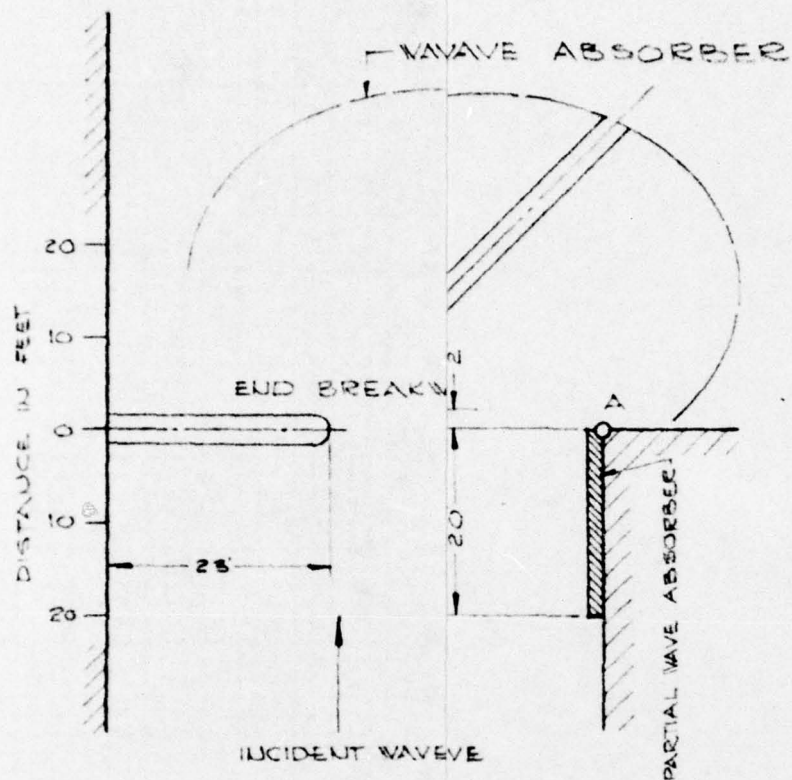
II-4. 2) The end of the breakwater has to be built out of the diffraction zone from the corner A (Figure 1).

Inevitable errors have to be expected for the normal section. They could be reduced by having a small wave absorber close to this corner (made of chicken wire, for example). The diffraction effect is distributed along the length of the absorbers. Then the



wave height is slowly reduced without sharp variation of wave height. The end of the breakwater will be on the same line as AB which is always out of the zone affected by diffraction.

II-4. 3) The following graph (Figure 2) gives the effects of diffraction for the longest wave period in the case where there is no lateral wave absorber. This corresponds to the worst operating conditions to be expected. The reader can also refer to references 2, 3 and 4.



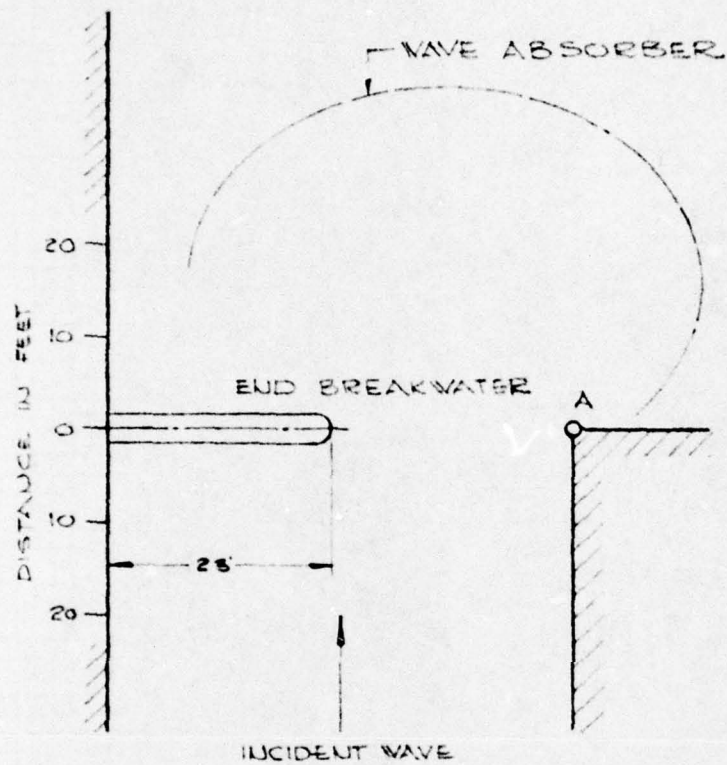
NORMAL ATAT 135°

FIGURE 1

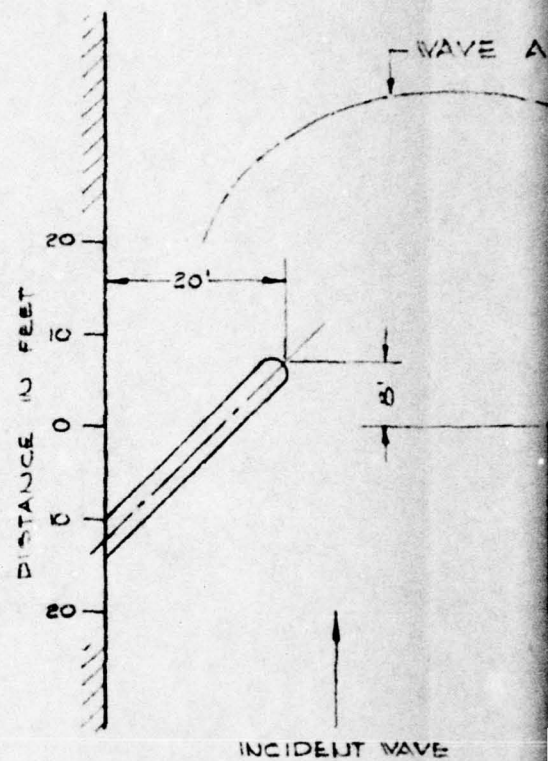
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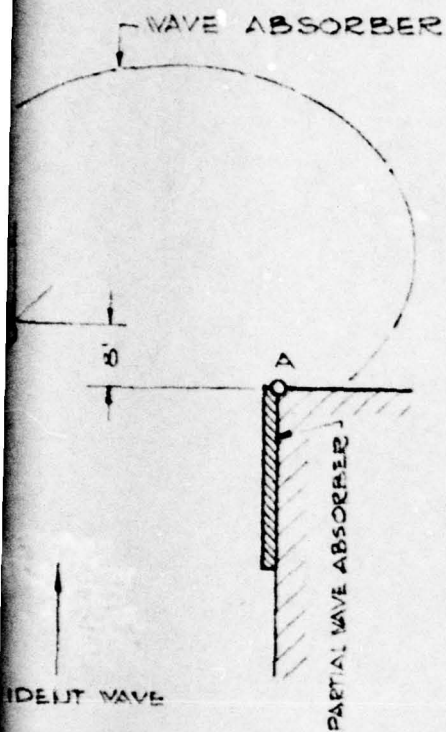
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ATTACK AT 45°

FIGURE 1

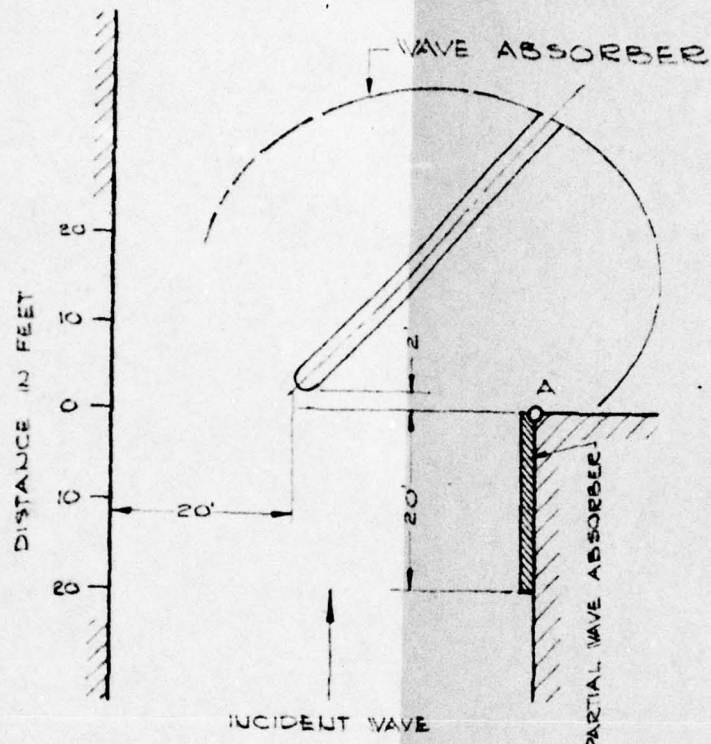
LAYOUTS FOR STUDIES OF



WAVE ABSORBER

INCIDENT WAVE

PARTIAL WAVE ABSORBER



WAVE ABSORBER

INCIDENT WAVE

PARTIAL WAVE ABSORBER

WAVE ABSORBER

NATIONAL ENGINEERING SCIENCE CO.

711 SOUTH FAIR OAKS AVENUE

PASADENA, CALIFORNIA

4-13-62

PB-3-1668

FEET

ERROR $\pm 1\%$

ERROR $\pm 1\%$

INCIDENT WAVE



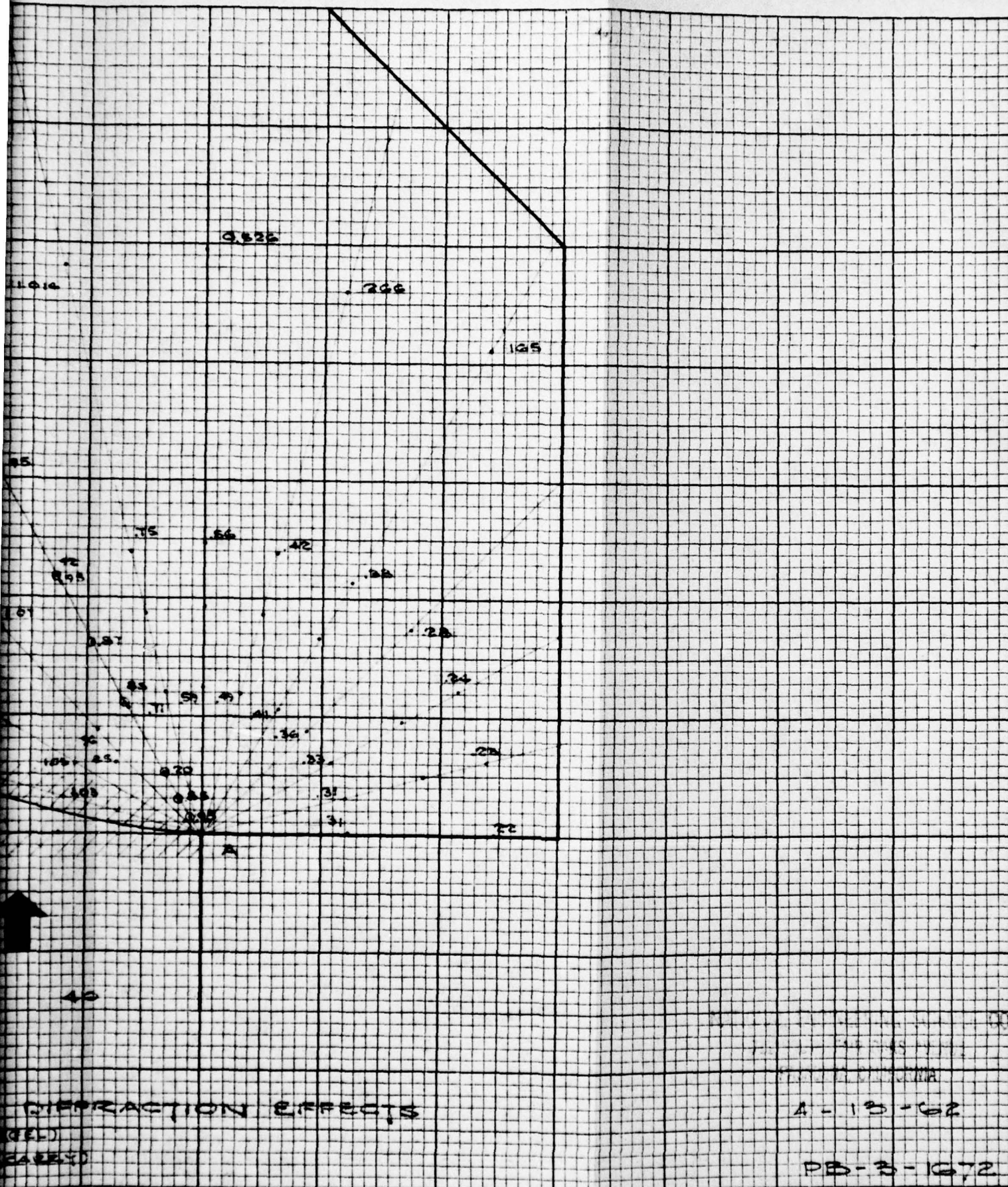
10 20 30
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FIGURE 2

WAVE BASIN - WAVE DIFF.

+ BENNY AND PRICE (WAGEL)

o PLYHAM AND ARTHUR (CASEY)



III. WAVE ABSORBERS

III-1) Wave Absorber Theory

III-1. 1) Wave absorbers are necessary at the two ends of the wave tanks. In both cases the best way to absorb wave energy is by causing the wave to break on a beach (as the best way of absorbing water power energy in a stilling basin is by causing a hydraulic jump).

The reflection coefficient depends upon the wave steepness and the slope, roughness and perviousness of the beach. In the case of a smooth inclined plane, the Miche formula⁽⁵⁾ can be used: The limit steepness of a wave which can give a total clapotis for a given value of the angle of the inclined plane with the horizontal is:

$$\gamma_{\max} = \frac{H}{L} = \sqrt{\frac{2\alpha}{\pi}} \frac{\sin^2 \alpha}{\pi}$$

Above this value there is breaking and the reflection coefficients R equals $\frac{\gamma_{\max}}{\gamma_0}$ where γ_0 is the wave steepness in deep water.

Because of the friction effect on the wall, the reflection coefficient can be reduced by a factor of 0.8 or 0.9, and as much as 0.5 because of roughness and perviousness.

The following graph (Figure 3) obtained experimentally by Greslau and Mahe⁽⁶⁾ can be used also to calculate reflection coefficients. It has been obtained with a smooth impervious wall. If the beach is pervious, the reflection coefficient is smaller by a factor of about 0.6.

It is seen, according to the Miche theory and Figure 3, that the reflection coefficient for a long wave of small amplitude is high unless the slope is very gentle. In this case it is preferable to have a wave absorber

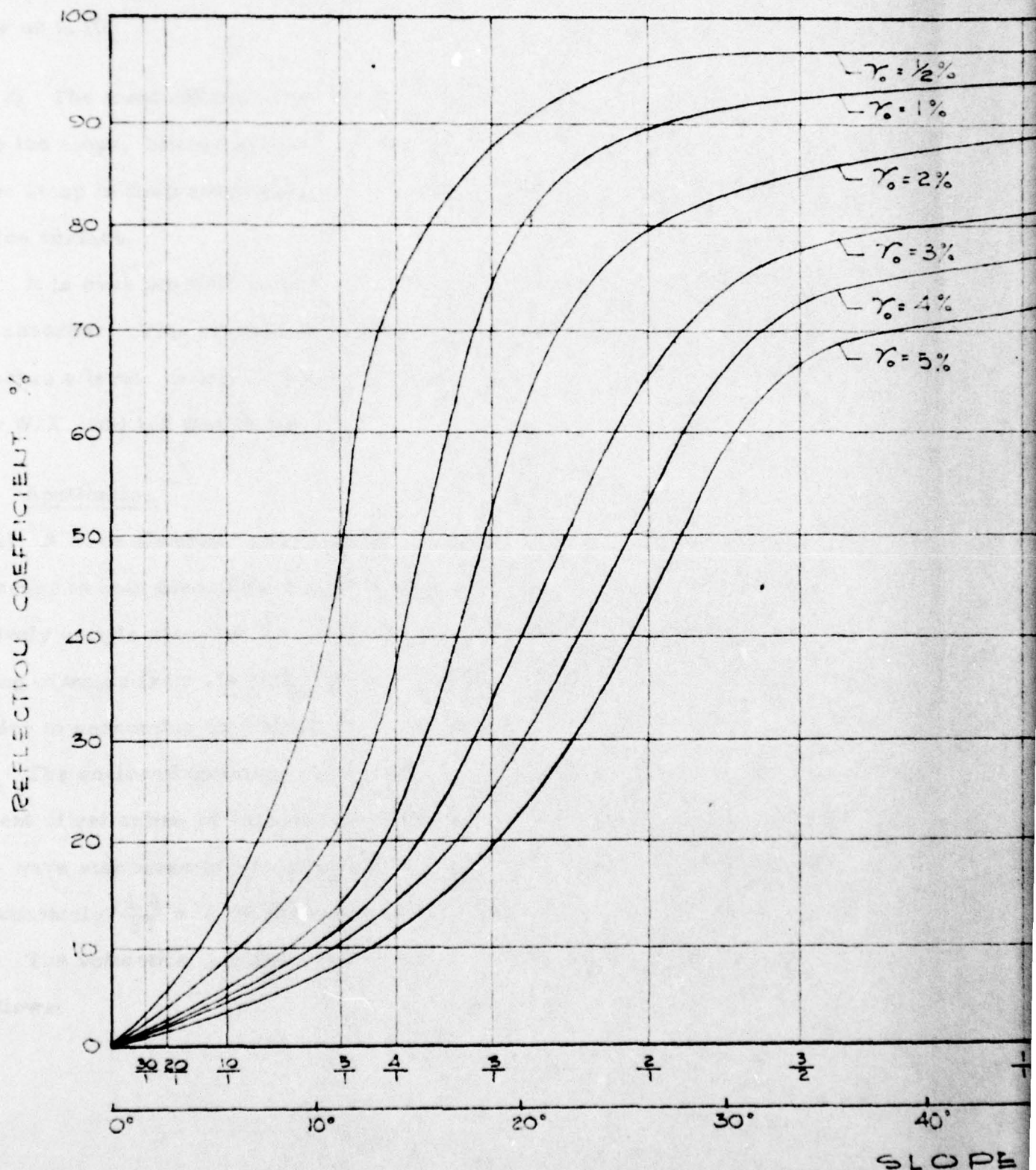
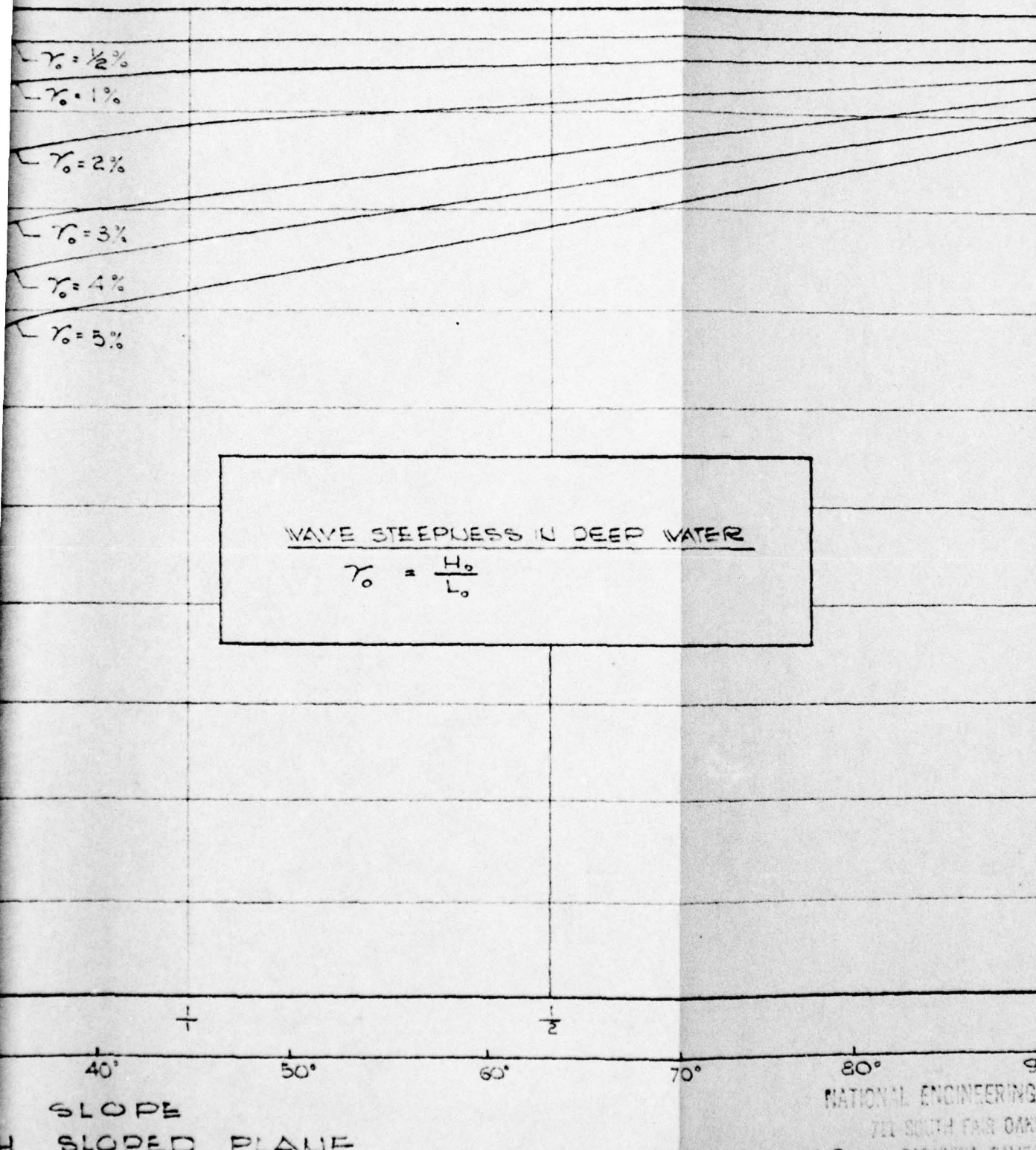


FIGURE 3 REFLECTION OF A SMOOTH SLOPED



SLOPE
H SLOPED PLANE

based on the progressive wave filter principles. For a progressive wave filter having a length equal to a wave length, the reflection coefficient could be as low as 0.10.

III-1. 2) The most efficient part of the wave absorber is the upper part. (7) Hence the slope, instead of being a constant, may be variable. The beach will be steep in deep water and gently sloped in the form of a parabola near the free surface.

It is even possible to have a vertical wall in the deeper part of the wave absorber. This vertical wall induces a very small reflection provided it reaches a level under $1.3 \times H$ under the free surface; that is, 2.6 feet in the W. F. and 1.3 feet in the W. B.

III-2) Application

III-2. 1) A wave absorber is necessary behind the wave paddle since waves are issued in both directions from the paddle. This wave absorber can be relatively simple since its only purpose is to avoid resonance and possible spilling of water from the tank. It may even be ended with a vertical wall in order to economize on length where it no longer has any function.

The enclosed drawings illustrate the principles involved. The coefficient of reflection of such wave absorbers may be calculated as a function of the wave steepness and average bottom slope. The average slope will be approximately $\frac{4.6}{27} = 0.16$ for the W. F. and $\frac{4}{18} = 0.22$ for the W. B.

The reflection coefficients as a function of the wave steepness are as follows:

	<u>H/L</u>	<u>0.05</u>	<u>0.005</u>
W. F.		0.06	0.25
W. B.		0.08	0.40

Even for the smallest steepness the above values are sufficient to avoid resonance and splash.

Furthermore, distances of about 20 feet and 30 feet in the W. B. and W. F., respectively, are sufficient to avoid an alternative rise in the mean water level between the paddle and the end wall.

III-2. 2) The wave absorbers at the far end of the flume must be quite efficient, particularly for the W. B. A distance equal to one wave length is required in order to promote a good energy absorber because the wave agitation passing between the structure and the side wall may be characterized by small steepness.

The slope of the beach near the M. W. L. being near 1/20, the wave reflection coefficient will be 0.08 for a wave steepness of 0.005.

III-2. 3) Several lateral wave absorbers may be required along the side wall of the W. B. in which case the best absorbers are those made of framed boxes of chicken wire filled with steel cuttings.

IV. WAVE FILTER

IV-1 Wave Filter Theory

IV-1. 1) Prior to the invention of the wave filter, it was necessary to build a long wave tank and limit the duration of the tests to a few minutes.

At the coast waves arrive from an infinite distance, so to speak, and when reflected by the shore or a structure, they return to an infinite distance. Finally the waves are slowly damped by friction.

In a wave tank the waves are issued from the paddle and when reflected by the structure under study, the waves are interrupted and reflected again by the paddle.

The wave filter, introducing a high friction force, permits reproduction of good boundary conditions which in effect replace the infinite ocean distance. Moreover, by its friction effect, the wave filter has a tendency to dampen primarily the harmonic components issued from the paddle. After passing through a wave filter, the wave motion is nearly perfectly monochromatic. This is important in studying the hydrodynamics of gravity waves. It is less important in coastal engineering since, conversely, waves at sea are highly irregular.

IV-1. 2) The first wave filter and wave filter theory are due to Biesel.⁽⁸⁾⁽⁹⁾ In fact, a study of O'Brien⁽¹⁰⁾ on the effect of wall friction on gravity waves may also be considered as a wave filter theory.

Briefly, it is demonstrated that when the equations are linearized and after introduction of a friction force proportional to V , the wave amplitude decreases with distance as $D = e^{-\frac{\Delta x}{G}}$ where G is the group

velocity and λ a coefficient of friction. It is seen that the damping effect increases when G decreases for a shorter period. This explains why the shortest components of the motion have a tendency to disappear.

IV-1.3) In the theory of Biesel, the wave filter is assumed to be immaterial, that is, void parameter equal to zero. NESCO has had the opportunity of continuing the theory of Biesel for a filter of void parameter ϵ (ϵ = ratio of volume of void full of water and total volume. This is also the case for pervious breakwater,⁽¹¹⁾⁽¹²⁾ whence $D = e^{-\frac{\epsilon \lambda x}{1.2 G}}$

In addition, a new wave filter, the "progressive wave filter," has been developed which is of great importance for absorbing long waves over a short distance.⁽¹³⁾

Finally, new additions have been made in regard to wave filter theories. Although these studies have not yet been published, the main results are presented here.

The reflection coefficient of a wave filter due to the introduction of friction force without change of cross section is

$$R = \frac{\lambda}{2k} \left[1 - \frac{1}{1 + \frac{2md}{\sinh 2md}} \right]$$

where $m = \frac{2\pi}{L}$ and $k = \frac{2\pi}{T}$. This relationship shows that by increasing the damping effect, one increases the reflection of the filter.

This shows the advantage of the progressive wave filter.

The mathematical theory of the progressive wave filter has been developed but is unpublished to date. The length of the equations obtained

does not justify their inclusion in this report but they present more of an academic interest because one is not able to relate theoretically the friction coefficient λ to the system used as a filter.

From a more practical point of view, one has to solve two problems:

- (1) What damping effect is necessary
- (2) How to realize it in practice

IV-2) Calculation of the Required Damping Effect

IV-2. 1) The damping effect to be inserted in the wave tank by the wave filter depends upon the kind of structure to be investigated. The following short theory is unpublished, so it has been judged useful to insert it in this report.

Consider a wave of amplitude unity issued from the wave paddle. (Figure 4) This wave becomes D after passing through the filter. The corresponding incident wave height on the structure is D . A part of its energy is destroyed on the structure, which is assumed to have a reflection coefficient β_1 . Hence, the wave coming back to the filter has an amplitude of $D \beta_1$. This wave passing through the filter becomes $D^2 \beta_1$. If β_2 is the reflection coefficient of the paddle, this wave becomes $D^2 \beta_1 \beta_2$ and passing through the filter once more becomes $D^3 \beta_1 \beta_2$. This wave component is superimposed to the initial wave of wave height D . They can have any phase difference. At the limits they are in phase; then the resultant wave is $D + D^3 \beta_1 \beta_2$. Or they can be in complete opposition of phase; then the resultant wave is $D - D^3 \beta_1 \beta_2$.

The fact that the waves are in phase or in opposition of phase depends upon the number of wave lengths between the paddle and the structure. They will be

WAVE PADDLE

WAVE FILTER

BREAK WATER

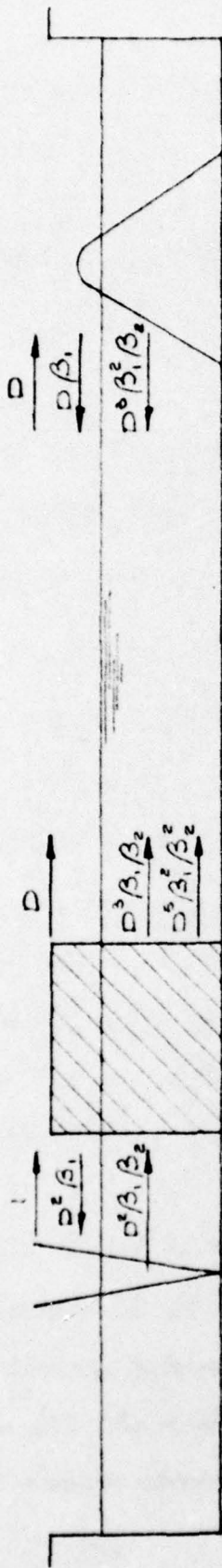


FIGURE 4
NOTATION FOR CALCULATING A WAVE FILTER

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in phase if this number of waves is $n \frac{L}{2}$ where n is an integer and L the wave length. They will be in opposition of phase if this distance is $n \frac{L}{2} + \frac{L}{4}$.

Now consider the component $D^3 \beta_1 \beta_2$ which, reflected by the structure, becomes $D^3 \beta_1^2 \beta_2$. Then $D^3 \beta_1^2 \beta_2^2$, then $D^4 \beta_1^2 \beta_2^2$, and so on.

Finally the resultant wave, traveling from the filter to the structure, has a wave height which varies between:

$$D(1 + A + A^2 + A^3 + \dots) = \frac{D}{1 - A}$$

$$D(1 - A + A^2 - A^3 + \dots) = \frac{D}{1 + A}$$

depending on the distance between the paddle and the structure in terms of wave length. ($A = D^2 \beta_1 \beta_2$)

It is seen that when this distance is $n \frac{L}{2}$ and the incident wave amplitude on the structure is $\frac{D}{1 - A}$, there is resonance. If A is close to unity, that is $D = 1$ (no filter), β_1 close to unity (which is the case of a vertical breakwater), and β_2 close to unity (which is always the case), the wave amplitude varies from $\frac{1}{2}$ to infinity when the distance between the wave paddle and the vertical breakwater in terms of wave lengths varies from $n \frac{L}{2} + \frac{L}{4}$ to $n \frac{L}{2}$.

In reality a definite state of agitation on such a long distance never exists. The fluctuation of the wave period T due to electrical or mechanical variations can cause the motion in the tank to be very unstable and irregular. This is due to the fact that a slight variation of L due to a slight variation of T causes the number of wave lengths in the tank to be either $n \frac{L}{2}$ or

$n \frac{L}{2} + \frac{L}{4}$. This is particularly frequent for short waves since in that case, n is large.

IV-2. 2) The previous considerations permit us to calculate the value of D for the wave height to stay within a given range of amplitude, whatever the fluctuation of period due to the wave generator.

In the case of a vertical breakwater, β_1 is chosen to be 0.8. Then, with $\beta_2 = 0.9$ and $D = 0.5$, $A = (0.5)^2 \times 0.9 \times 0.8 = 0.18$. The range of variation of wave amplitude in the most unfavorable case is:

$$\frac{1}{1+0.18} = 0.8 \qquad \frac{1}{1-0.18} = 1.2$$

In the case of a rubble mound breakwater with $\beta_1 = 0.3$, for example, it is sufficient to have $D = 0.80$ to obtain the same range of wave amplitude.

In the case of a study of beach equilibrium with $\beta_1 = 0.1$, for example, D could be as high as 1.0, making the use of a wave filter unnecessary.

As a conclusion, for normal studies in the wave tanks, $D = 0.80$ could be considered as a normal operating value. However, it may be wise to reserve the possibility of having a value of D as low as 0.5.

Various papers have been presented which give the value of the reflection coefficient of structures. The combination of these results with the calculations presented in this report permits evaluation of the best damping coefficient. However, in practice, the best method is trial and error since one is not able to accurately relate D with the filter length.

IV-3) Wave Filter Construction and Operation

IV-3. 1) A solution is required to the problem of introducing friction within the wave tanks. Various methods are used in various laboratories to solve this problem but very few quantitative results are available. The best system consists of using vertical perforated planes. The main advantage of this solution is that the planes operate not only as filters but also as guides to avoid transverse resonance. This solution permits one to decrease slightly the length of the tanks and the facility in which they are built. However, a plane at least every inch of length is required to have a good damping effect. Hence, this solution is so costly that it has been abandoned.

Other possible solutions exist in using chicken wire or by compressing aluminum or wood shavings in framed boxes made of chicken wire.

IV-3. 2) The solution which is adopted here consists of having cloth or plastic suspended vertically from horizontal bars, perpendicular to the wave travel. This solution has many advantages. It is the cheapest method; the wave filter can be removed easily; the damping coefficient can be adjusted carefully; the elements can be spaced widely at the limit of the filter and closely at the center; and so on. The elements act as a progressive wave filter.

IV-3. 3) When the wave paddle is stopped because of the inertia of the fly wheel, waves of longer period are issued from the paddle. These waves, traveling more quickly than the operating wave, have a tendency to be superimposed to these operating waves in front of the structure. Hence, the last moment of the test is often a cause of great change in the equilibrium profile of the structure and beaches.

Because of this effect, it is often useful to stop the wave issued by the paddle before stopping the paddle itself. This can be done by dropping more filter elements of very great rigidity, i. e. with a heavy loading rod at the bottom. These elements should be suspended above the water level during the test.

IV-3. 4) The necessary total length of wave filter depends upon the efficiency of these elements. No quantitative data is available but it is unanimously considered that half a wave length is required to absorb enough energy without reflection in a filter. Hence, a length of 35 feet of filters will be required in the W. F. (20 ft. before the convergent and 15 ft. after the convergent). No filters have been introduced in the convergent in order to have the filter elements of identical width and depth. However, this could be an alternate solution without any hydraulic disadvantage.

In the W. B. more room is available along the 50 ft. sloped bottom. Hence the wave filter may conveniently be located at this place. The vertical length has been calculated to be 9 ft. and 7 ft. for the W. F. and 5 ft. 6 in. for the W. B.

SECTION TWO

WAVE PADDLES

I. A REVIEW OF WAVE PADDLE THEORIES

I-1) Linear Theory, Sinusoidal Motion of the Paddle

The wave paddle theory was established by Havelock in 1929⁽¹⁴⁾ and refined by Biesel in 1951.⁽¹⁵⁾ Biesel demonstrated the validity of the solution proposed by Havelock. Kravtchenko completed and finished the demonstration made by Biesel.⁽¹⁶⁾

In all cases it was assumed that the wave paddle had a sinusoidal motion. Havelock and Biesel found that the wave motion issued by the paddle is given at a first order of approximation by a single periodical gravity wave motion and an infinity of sinusoidal oscillations of the same period. These sinusoidal oscillations are given in the form of a series of an infinite number of terms. They disappear exponentially with the distance from the paddle. In practice they are negligible at a distance equal to two or three times the depth of the wave basin.

I-2) Linear Theory, Non-Sinusoidal Motion of the Paddle

The motion of the wave paddle is periodical but not always exactly sinusoidal. A slight variation of angular velocity exists due to the fact that the force on the motor is not constant but varies because of the inertia of the paddle, the connecting rod and the crank, and the variation of the hydraulic pressure on the paddle.

The wave motion far from the paddle is given to a first approximation by a Fourier series representing a fundamental wave and harmonics. At the paddle the motion is periodic, having the period of each of these harmonics as well as a local oscillation which disappears with distance. This demonstration was carried out by Apte (unpublished). The solution to this problem is obtained by use of a heavy fly wheel connected to the wave generator. The wave filters also have a tendency to dampen harmonic motion more than the fundamental wave motion.

I-3) Non-Linear Theory. Sinusoidal Motion of the Paddle

All the previous theories are linear. They do not explain the presence of harmonic motion even when the motion of the wave paddle is rigorously sinusoidal. These harmonics are due to nonlinear effects. It has been shown at a second order of approximation by Fontanet⁽¹⁷⁾ that harmonic motion exists with a period $T/2$ (T being the period of the fundamental) which is superimposed to the principal wave motion. Near the paddle a local oscillation of period $T/2$ of second order also exists and disappears with distance.

Far from the paddle the wave motion due to a perfectly sinusoidal motion of the paddle could be considered at a second order of approximation as the superimposition of:

(1) A fundamental sinusoidal wave motion of period T . This is a free wave, the motion of which is the same as that obtained at first order of approximation.

(2) A component having a period $T/2$ which may be considered as a wave having a height proportional to the square of the wave height given by (1) above. This is a forced wave having the same wave celerity as the wave of period T .

(3) A harmonic of period $T/2$, also of amplitude proportional to the square of the wave height of wave (1), but has its own wave celerity which is that of a free wave of period $T/2$.

Other special wave paddle theories have been carried out such as the "snake paddle" theory of Biesel. (18)

For practical purposes only the linear theory due to a paddle having a sinusoidal motion will be used in the following. This theory is quite sufficient for designing wave paddles. The other theories are more of academic interest. Our main source of reference will be the work done by Biesel. (15)

II CALCULATION OF WAVE HEIGHTS

II-1) Linear Wave Paddle Theory

It is not the purpose of this report to deal with wave paddle theory but rather to apply it. However, it is useful to recall the basic assumptions on which our calculations will be based.

(1) The fluid is assumed to be perfect and there is no leakage at the end of the paddle. Hence, an empirical efficiency factor smaller than one must be inserted in the final results.

(2) The convective inertia term is neglected; i. e. the theory is no longer valid for very steep waves in shallow water. The theory is exact for small wave motion.

(3) The flow is assumed to be irrotational.

It is then found that the theoretical amplitude of the wave H_{th} issued by the paddle is linearly related to the stroke $\xi(z)$ by the general relationship:

$$H_{th} = \frac{4 m \sinh m d \int_0^d \xi(z) \cosh m z dz}{\sinh m d \cosh m d + m d}$$

where $m = \frac{2\pi}{L}$ (L is the wave length)

d = depth at the paddle

z = distance from the bottom

$\xi(z) = \frac{1}{2}$ amplitude of the stroke of the paddle at various depths.

In the case of a piston wave paddle, $\xi(z) = e = \text{cst}$; then,

$$H_{th} = \frac{2 \sinh^2 m d}{\sinh m d \cosh m d + m d} 2e = K \cdot 2e$$

In the case of a paddle hinged on the bottom, $\xi(z) = \frac{e}{d} z$, where e is the stroke at the free surface. Then:

$$H_{th} = \frac{2 \sinh md (1 - \cosh md + md \sinh md)}{md (\sinh md \cosh md + md)} \times 2e = K' \times 2e$$

K and K' as defined above are a function of md only or L/d only. Figure 5 gives the value of K and K' as a function of L/d.

This graph is the fundamental data on which the following calculations are based. It may be observed that for large values of L/d a piston paddle of stroke $2e = 2$ feet, for example, will give the same wave height as a hinged paddle with a stroke of 4 feet at the free surface. For small values of L/d, the wave height tends to be the same for the same value of the stroke at the free surface.

II-2) Requirements

II-2. 1) It has been seen that the extreme requirements are:

	Wave Period T (sec.)	Minimum Depth (ft.) d_t	Maximum Wave Height (ft.)
W. F.	4.2	4	2.5
W. B.	3	2	1.25

The corresponding values for L/d, K and K' are (without filter):

	Wave length (ft)	Depth (ft)	L/d	K	K'
W. F.	45 say 50	4	12.5	0.26	0.48
W. B.	25	2	12.5	0.26	0.48

II-2. 2) Calculation of the Stroke at the Free Surface

The efficiency η of the wave paddle is defined by the relationship $H = \eta 2e K$ or $H = \eta 2e K'$ where H is the factual wave height given by the paddle. This efficiency is slightly greater for a hinged paddle

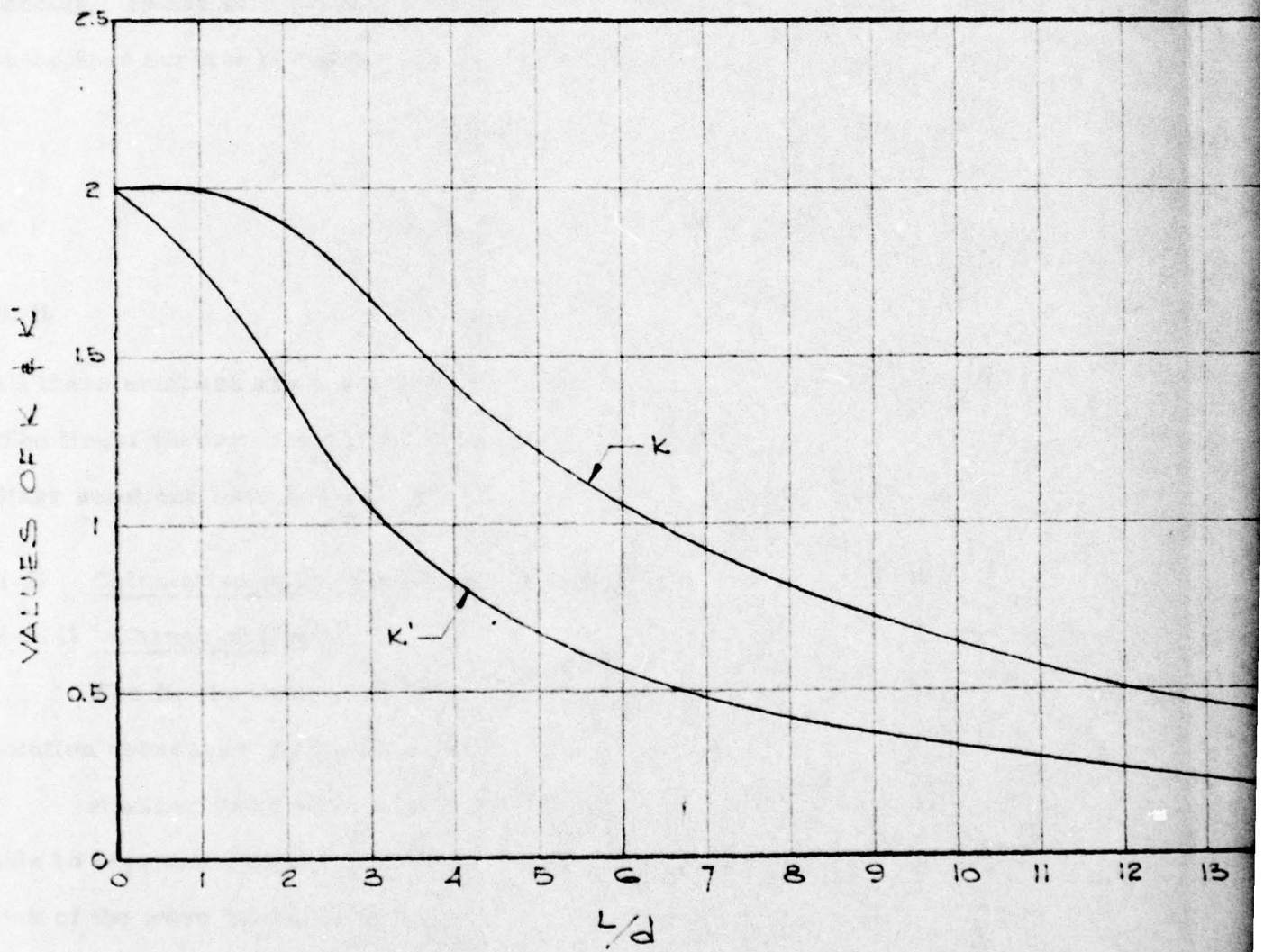
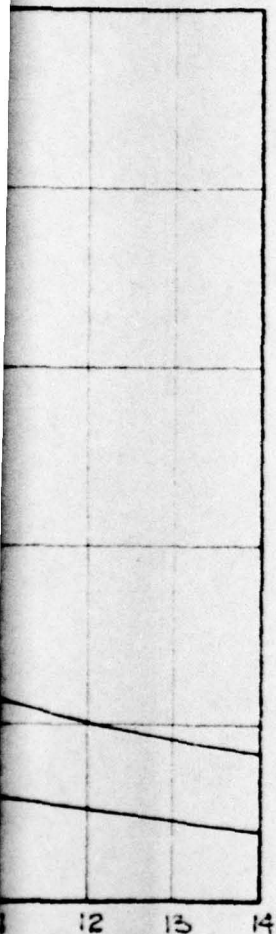


FIGURE 5

STANDARD FOR WAV



$$K' = 2 \frac{\sinh md (1 - \cosh md + md \sinh md)}{md (\sinh md \cosh md + md)} : \text{HUGED TYPE}$$

$$K = 2 \frac{\sinh^2 md}{\sinh md \cosh md + md} : \text{PISTON TYPE}$$

$$L = \text{WAVE LENGTH IN FT. } (L = \frac{gT^2}{2\pi} \tanh \frac{2\pi d}{L})$$

$$d = \text{DEPTH IN FT.}$$

$$T = \text{PERIOD IN SEC.}$$

$$m = 2\pi/L$$

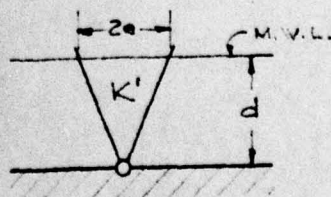
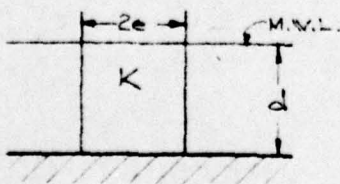
$$H = \eta (K \text{ OR } K') 2e$$

$$H = \text{WAVE HEIGHT IN FT.}$$

$$\eta = \text{EFFICIENCY (0.8)}$$

$$2e = \text{STROKE AT THE FREE SURFACE IN FT.}$$

$$\frac{H}{L} < 0.12 \tanh \frac{2\pi d}{L} \text{ OR } \frac{H}{d} < 0.7$$



NATIONAL ENGINEERING SCIENCE CO.
711 SOUTH FAIR OAKS AVENUE
PASADENA, CALIFORNIA

FOR WAVE PADDLE CALCULATION

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than it is for a piston type paddle because of the leakage between the bottom and the paddle. However, it will be considered that the efficiency is the same and equal to 0.8 as it has been obtained by some experiments on small paddles. Hence this number is conservative. Then the required stroke $2e$ at the free surface becomes:

$$2e = \frac{H}{\gamma \times K \text{ or } K'}$$

	2e Piston Type (ft)	2e Hinged Type (ft)
W. F.	$\frac{2.5}{0.8 \times 0.48} = 6.5$	$\frac{2.5}{0.8 \times 0.26} = 12$
W. B.	$\frac{1.25}{0.8 \times 0.48} = 3.25$	$\frac{1.25}{0.8 \times 0.26} = 6$

All these numbers are mechanically as well as hydraulically unrealistic.

(The linear theory of wave paddle does not hold true for such an amplitude.)

Other solutions have to be found.

II-3) Calculation of the Maximum Stroke and General Arrangement

II-3.1) Change of Depth

The first solution exists in increasing the depth at the paddle. This solution decreases L/d , hence increases the values for K and K' .

Another great advantage of increasing the depth at the paddle is being able to reproduce waves close to the limit steepness in the test area without risk of the wave breaking at the paddle.

A local agitation near the paddle is due to the fact that the paddle does not produce a wave which is exactly cylindrical. Hence it often occurs that the wave breaks at the paddle despite the fact that the condition for limit steepness is not fulfilled. These conditions are, for example:

$$\left(\frac{H}{L}\right)_{\max} = 0.12 \tanh \frac{2\pi d}{L} \quad (\text{Modified Miche formula})$$

or $\frac{H}{d} = 0.70$ (Modified solitary wave theory for long wave in shallow water)

If the wave breaks at the paddle, it is more difficult to obtain a steep wave in the test area.

II-3.2) Multiplication Factor Due to Change of Depth

Moreover, the change of depth may cause an increase of wave height which may be calculated by applying the principle of conservation of energy: $H^2 G = \text{cst}$ where G is the group velocity. For long waves in shallow water, which is the case encountered for the maximum condition, the increase in height from the paddle to the test area is proportional to $\sqrt[4]{\frac{d}{dt}}$ where d is the depth at the paddle and dt the depth in the test area.

After other considerations, the minimum depth at the paddle has been chosen to be 6 ft. in the W. F. and 4 ft. in the W. B. The multiplication factor for long waves due to the change of depth becomes:

$\sqrt[4]{\frac{6}{4}} = 1.11$ for the W. F. and $\sqrt[4]{\frac{4}{2}} = 1.2$ for the W. B. It is seen that this multiplication factor compensates the wave filter effect in the W. B. Hence it is now sufficient to have a wave height at the paddle of the W. B. of 1 ft., and 0.83 ft. without a wave filter.

II-3.3) Multiplication Factor Due to Change of Width

It is very desirable to have steep waves in the W. F. in order to investigate experimentally the hydrodynamics of wave motion. In such a narrow flume, it is then possible to combine the change of depth with a convergent.

Applying the principle of conservation of energy, it is seen that the wave height at the paddle H and the wave height in the testing area H_t are related by: $H^2 G \ell = H_t^2 G_t \ell_t$ where G is the group velocity and ℓ the width of the flume. Index t refers to the testing area. For a long wave in shallow water, this formula gives:

$$H^2 \ell \sqrt{d} = H_t^2 \ell_t \sqrt{d_t}$$

The multiplication factor $\sqrt{\ell/\ell_t}$ due to the change of width is commonly known as the refraction coefficient. ℓ has been chosen equal to 10 ft., whence $\sqrt{\frac{10}{4}} = 1.58$. The combined effect of the change of depth and the change of width in the W.F. yields a multiplication factor for long waves of $1.11 \times 1.58 = 1.75$.

The required wave height at the paddle is reduced to $2/1.75 = 1.14$ with no filter and $1.14 \times 0.8 = 0.91$ with a wave filter.

II-3.4) Calculation of the Stroke with Multiplication Factors

These new requirements permit selection of a hinged paddle rather than a piston type paddle. This kind of paddle is mechanically simpler and less expensive, even if one includes the cost of a convergent which actually has been seen to be very useful.

Now, the value L/d , K and the stroke $2e$ at the free surface become:

	Wave Period T (sec.)	Depth (ft)	Wave Length L (ft.)	L/d	K'	Wave Height H_{th} (ft.) *
W. F.	4.2	6	54	9.00	0.35	1.14
W. B.	3	4	31	7.76	0.39	0.83

*without filter

	Efficiency	2e (ft)
W. F.	0.8	4.1
W. B.	0.8	2.66

Considering that some wave filter may always be necessary after the convergent in the W. F., the following values are finally adopted:

	Depth (ft.)	Stroke (ft.)	Tang θ *
W. F.	6	4.5	0.37
W. B.	4	2.33	0.29

$$*(\text{tang } \theta = e/d)$$

II-3.5) Maximum Wave Heights vs. Wave Period

It would be mechanically unrealistic to have a greater stroke. If any filter is used, smaller wave heights will be obtained but the wave height produced by the paddle increases quickly when the wave period decreases slowly. It is difficult to produce a long wave of great amplitude. It is easier to produce short waves of great amplitude up to limit steepness.

Some calculated results illustrate these considerations, assuming a wave filter with a damping effect of $D = 0.8$.

	Maximum Stroke 2e	Wave Height after filter H_t	Wave Height at Paddle H ($D = 0.8$)	Efficiency	Theoretical H
W. F.	4'6"	2'	$\frac{2}{0.8 \times 1.75} = 1.43$	0.8	1.8
W. B.	2'4"	1'	$\frac{1}{0.8 \times 1.11} = 1.12$	0.8	1.4
	K	L/d	d	L paddle	T
W. F.	0.4	7.7	6	46	3.8 sec.
W. B.	0.6	5.7	4	23	2.7 sec.

It is seen that these values are very close to the maximum requirements despite the wave filter effect.

II-3. 6) Figure 6 has a general value valid for any kind of wave paddle hinged at the bottom. It gives the wave height as a function of the wave period, the depth, and the stroke $\tan \theta = e/d$. It is easy to relate $\tan \theta$ to the eccentricity at the crank for a given installation. This nomograph includes the coefficient of efficiency and a wave damping effect D through the filter. Moreover, it permits one to determine the conditions when the wave breaks at the paddle.

II-4) A General Formula for Calculating the Wave Height in a Wave Tank

Taking all the previous phenomena into account, the wave height in the testing area may be calculated by taking into account:

- (1) The theoretical wave height at the paddle, which is a function of the period T , the depth at the paddle d , and the stroke $\tan \theta$.
- (2) The efficiency of the paddle η (0.8)
- (3) The wave filter effect $e^{-\frac{\lambda x}{G}}$ which is a function of the wave period T , the friction coefficient λ and the length of the filter x . (λ could be replaced by $\frac{1}{N}$ where N is the number of wave filter elements and Γ a coefficient experimentally determined of one element of wave filter.)
- (4) The change of depth from the wave paddle to the testing area by inserting the multiplication factor $\frac{G}{G_t}$ where G is the group velocity function of the period and depth. Since for a given wave tank

See Pocket in Rear Cover for
Figure 6.

$d = d_t + \Delta d$ where $\Delta d = \text{constant}$, this multiplication factor could be expressed as a function of T and d (or d_t) only.

(5) The change of width in the case of convergence, the multiplication factor being $\sqrt{\frac{1}{1_t}}$ and 1_t being the width in the testing area and 1 the length of the paddle.

Finally the wave height in the testing area for a given wave tank is given as a function of T , d_t , $\tan \theta$, and N . All the other parameters are fixed or determined from these four variables. However, λ (or f) and γ are fixed but unknown. They may be determined experimentally by a preliminary test.

The following set of formulas gives $H_t = f(T, d_t, \theta, N, \lambda, \gamma)$

$$H_t = \gamma H_{th} \sqrt{\frac{G}{G_t} \frac{\ell}{\ell_t}} e^{-\frac{\gamma N}{G}}$$

$$H_{th} = 2 K' d \tan \theta$$

$$K' = 2 \frac{\sinh md (1 - \cosh md + md \sinh md)}{md (\sinh md \cosh md + md)} \quad (\text{Hinged type})$$

$$m = \frac{2\pi}{L}$$

$$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi d}{L}$$

$$\frac{G}{G_t} = \frac{\tanh \frac{2\pi(d_t + \Delta d)}{L} \left[1 + \frac{\frac{4\pi(d_t + \Delta d)}{L}}{\sinh \frac{4\pi(d_t + \Delta d)}{L}} \right]}{\tanh \frac{2\pi d_t}{L} \left[1 + \frac{\frac{4\pi d_t}{L}}{\sinh \frac{4\pi d_t}{L}} \right]}$$

$$d = d_t + \Delta d \quad \begin{cases} (\Delta d = 1.5 \text{ ft in W. B.}) \\ (\Delta d = 2.0 \text{ ft in W. F.}) \end{cases}$$

This set of formulas has been presented in two nomographs (Figures 7 and 8) for the two wave tanks without wave filters. These formulas are not valid if the wave breaks. The wave can break at the paddle if $\frac{H}{d} > 0.12 \frac{L}{d} \tanh \frac{2\pi d}{L}$, or in the testing area if $\frac{H_t}{d_t} > 0.14 \frac{L_t}{d_t} \tanh \frac{2\pi d_t}{L_t}$ or $\frac{H_t}{d_t} > 0.78$ for a long wave in shallow water.

Figures 7 and 8 show the wave height in the testing area without wave filter effect as a function of T , d_t and $\tan \theta$ by taking into account the effect of the change of depth between the paddle and the test section and the convergent in the W. F.

$\tan \theta$ may easily be related to the eccentricity at the crank.

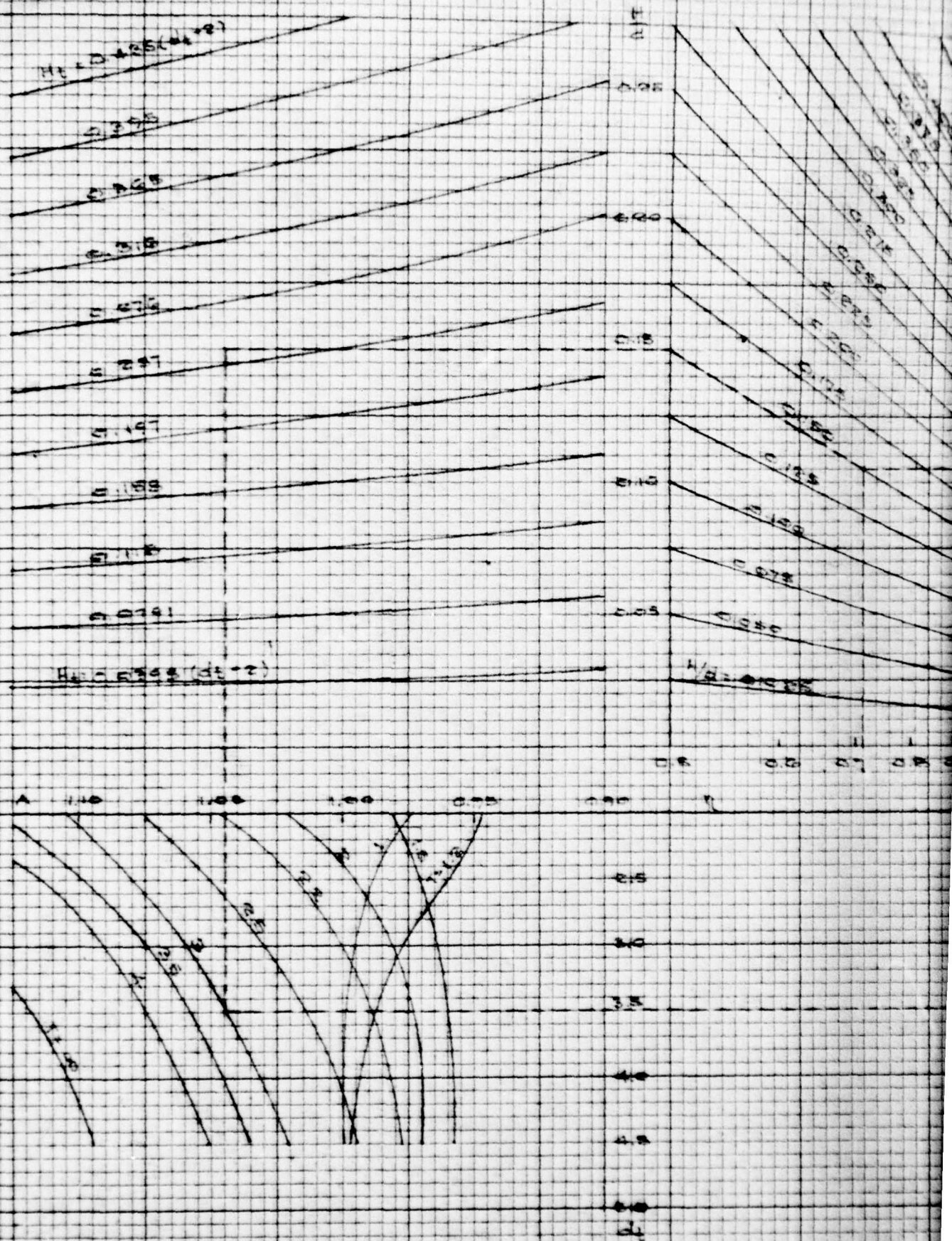
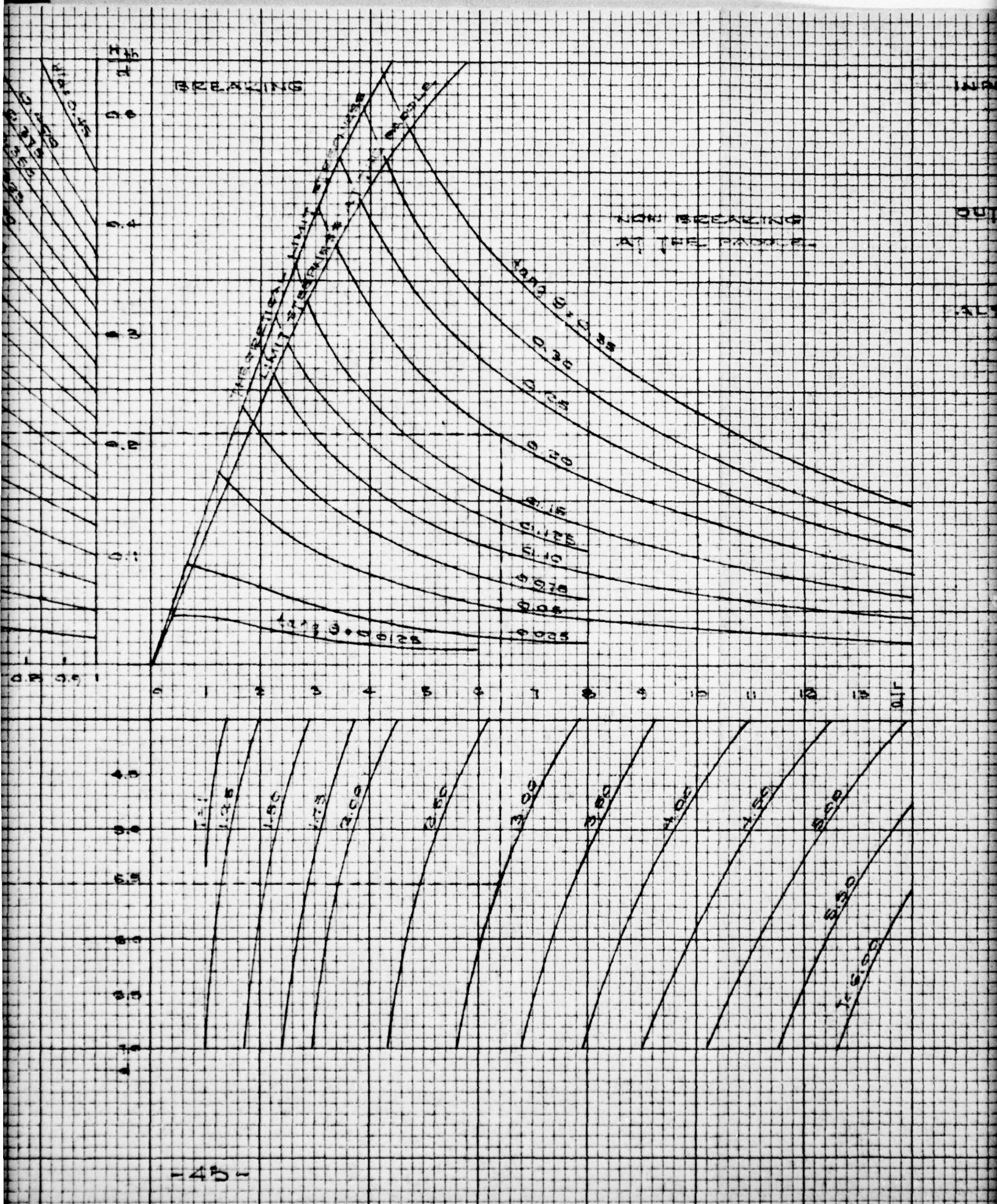


FIGURE 7
WAVE FLUME



INPUT:

$$Q = 5.5, H_L = 2.5 \text{ ft}$$

$$T = 3 \text{ sec}$$

$$\eta = 0.71$$

$$\tan \theta = 0.20$$

NOT BEARING
AT THE PADDLE

OUTPUT:

$$H_L = 0.250 (d_L + 2)$$

$$+ 0.250 (5.5)$$

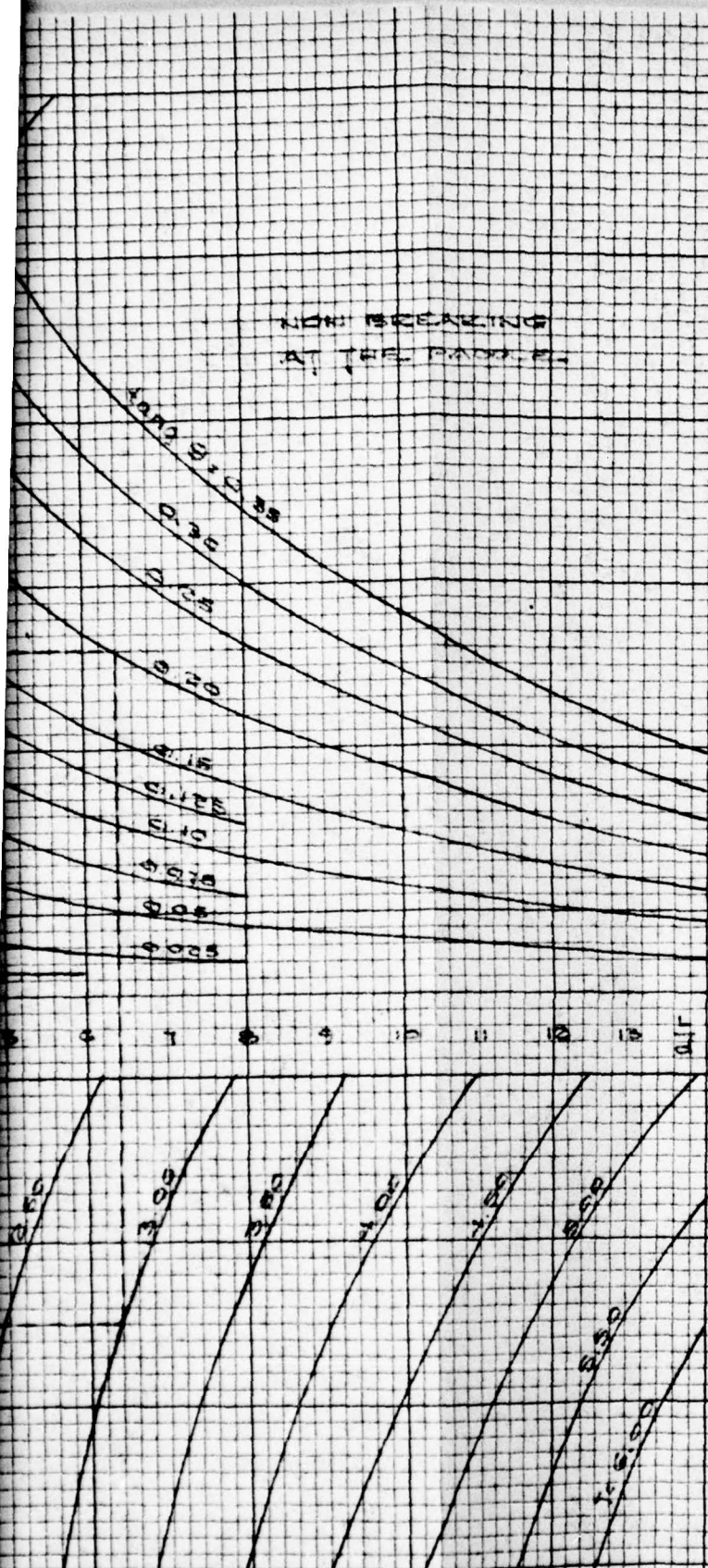
$$= 1.875 \text{ ft}$$

SLICE:

$$y/d = 0.4, L = 3.52 \text{ ft}$$

$$H_{up}/d = 0.21, H_{down} = 1.5$$

$$H/d = 0.15$$



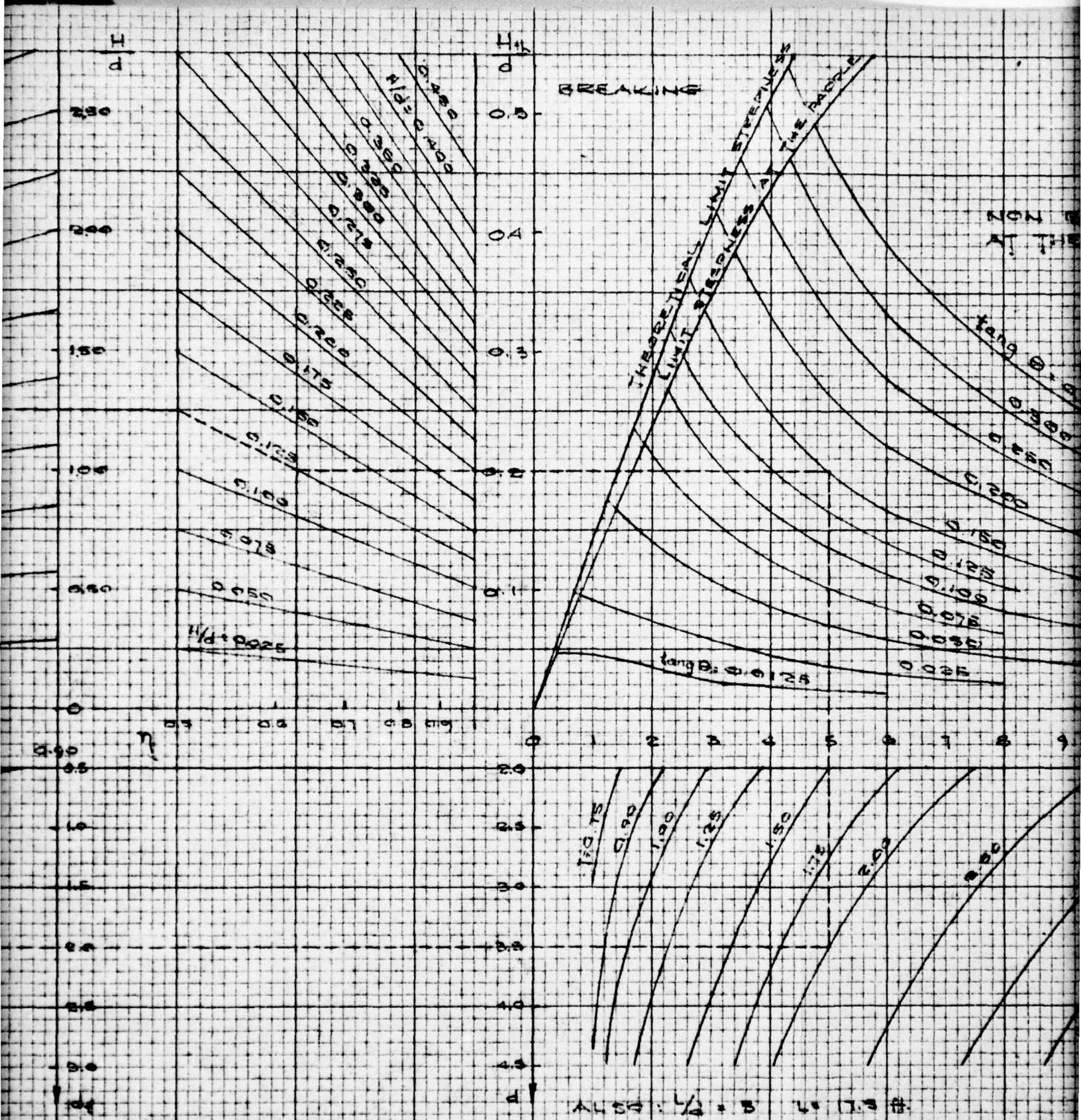
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BREAKING

THEORETICAL LIMIT STRESSNESS
LIMIT STRESSNESS AT THE PAOPLE

NON
AT THE

lang 0.025

0.075

0.100

0.125

0.150

0.200

0.250

0.300

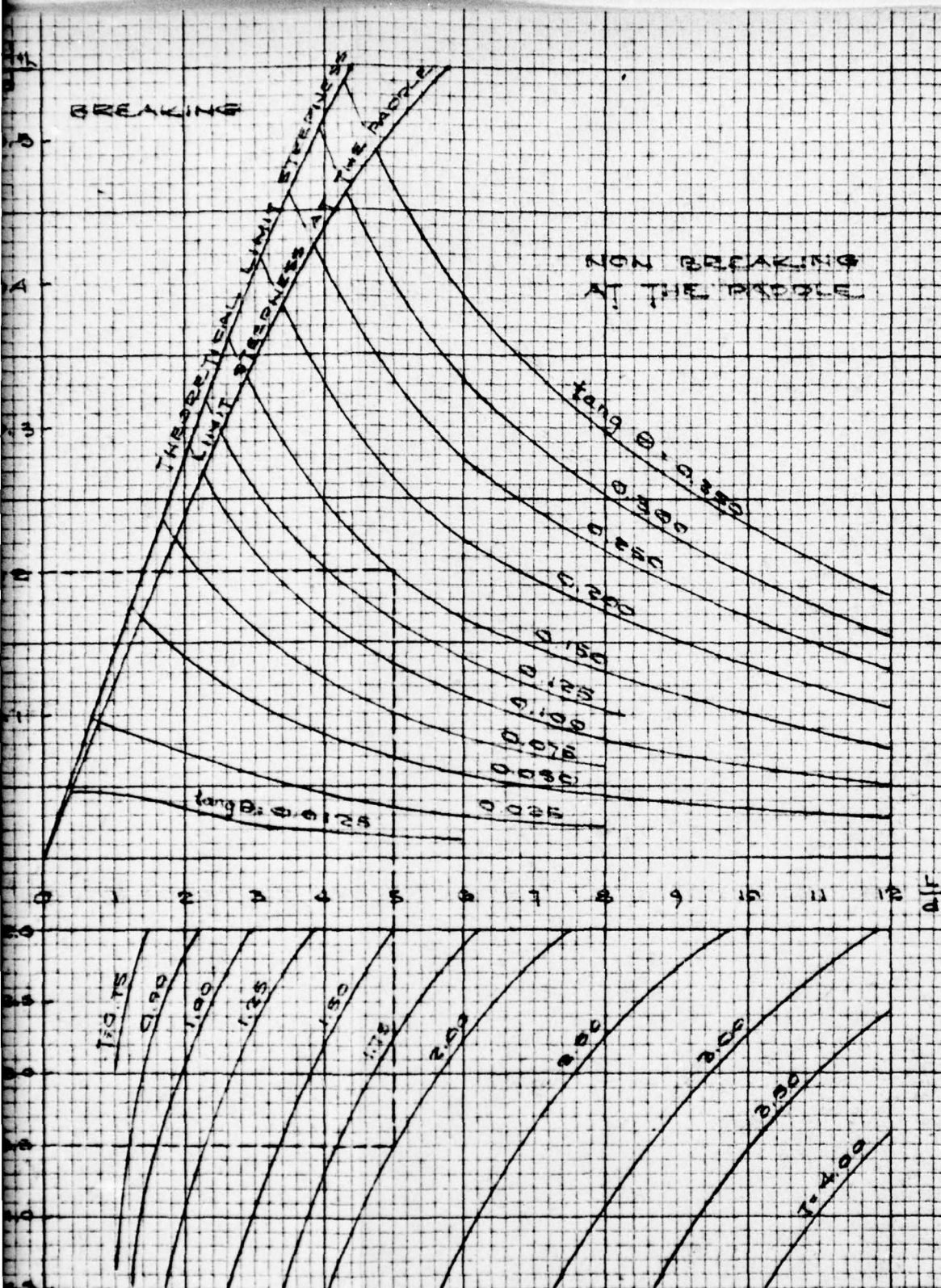
0.350

0.400

ALSO: $L/d = 5$ $L = 1.75$ ft.

$H_{th} = 5.20 d = 0.7$ ft THEORETICAL
 $H = 0.125 d = 0.44$ ft REAL H.
AMPLIFICATION FACTOR DUE TO

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ALSO: $L/2 = 5$ $L = 10$ ft.

$H_h = 0.20$ THEORETICAL AT THE PADDLE

$H = 0.125$ REAL H AT THE PADDLE

AMPLIFICATION FACTOR DUE TO CHANGE OF DEPTH APPROX

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III CALCULATION OF THE FORCES ON THE WAVE PADDLE AND POWER REQUIREMENTS

III-1) Forces on the Wave Paddle

The forces on the paddle are calculated from long equations. The pressure force may be considered as the sum of three terms. The first term is a hydrostatic pressure which acts equally on both sides of the paddle and does not need to be considered.

III-1. 1) Force Due to Wave Motion

A pressure force is due to the normal wave motion generated by the paddle. This pressure force is maximum when there is a crest on one side of the paddle and a trough on the other side. This occurs when the paddle is rigorously vertical and acts in a direction which is opposite to the motion of the paddle. This force is, according to the linear theory:

$$P_1 = 2l \int_0^d pw \, dz$$

where l is the length of the paddle (52') and $pw = \rho g \frac{H}{2} \frac{\cosh mz}{\cosh md} \cos kt$
 $= X_1 \cos kt$. For short waves this term is negligible by comparison with the third term p_i . (See III-1. 2, Section Two) For long waves it becomes:

$$[pw]_{\max} = \rho g \frac{H}{2} \quad \text{and} \quad P_1 = \rho g H d l$$

The pressure distribution tends to become hydrostatic and is distributed as shown in Figure 9, i. e. almost uniformly. Then it is found for the paddle of the wave basin that the corresponding maximum pressure force can be $62.4 \times 1.5 \times 3.5 \times 52 = 17,000$ lbs with $H = 1.5$ ft., and for the paddle of the wave flume, $62.4 \times 2.5 \times 6.5 \times 10 = 10,000$ lbs with $H = 2.5$ ft.

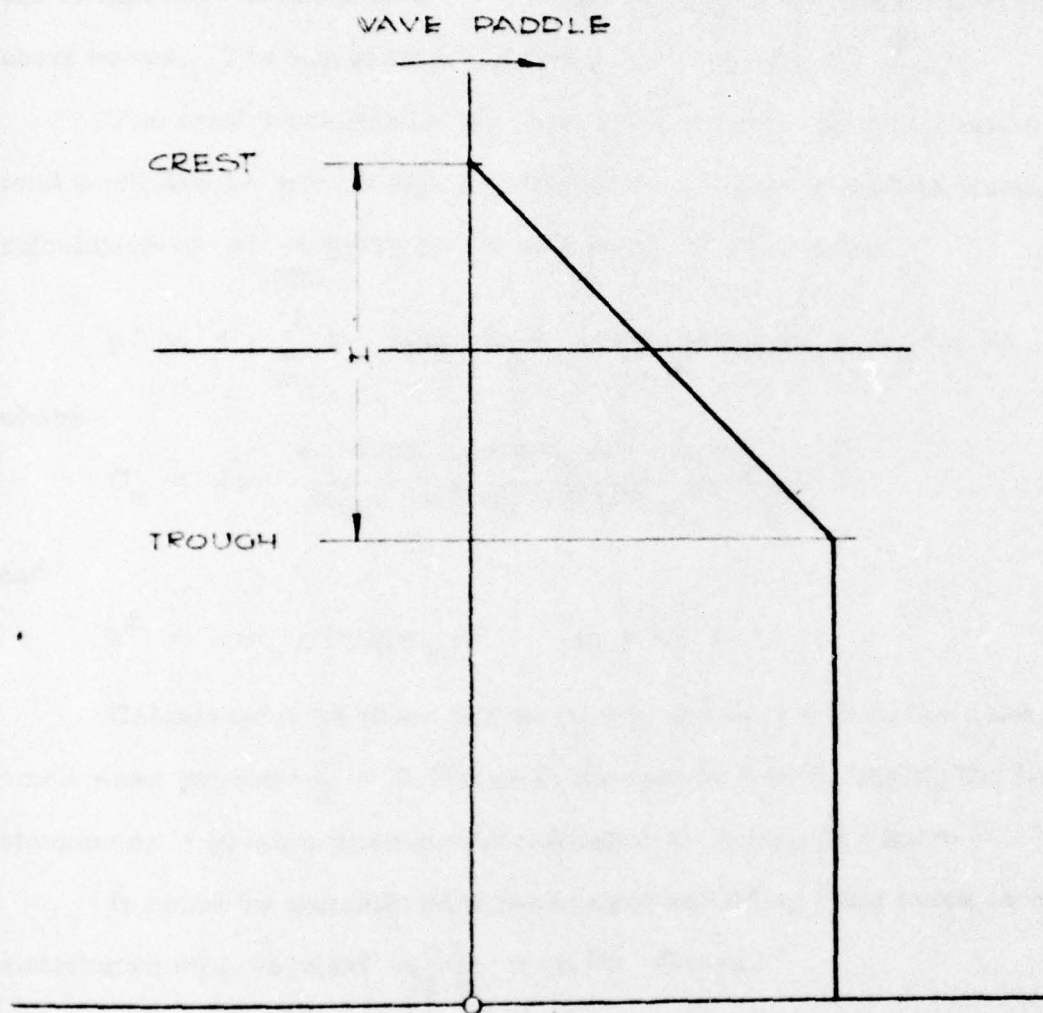


FIGURE 9
PRESSURE DISTRIBUTION ON THE PADDLE DUE TO LONG WAVES

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III-1. 2) Force Due to Water Inertia

The third force is given by $P_2 = 2\ell \int_0^d p_i dz$. This term is due to inertia. It tends to zero for long waves but becomes important for short waves. The lag of phase between p_i and p_w is $\frac{\pi}{2}$.

The total work due to p_i over a wave period is zero, while the total work due to p_w is equal to the work of the wave motion itself. The calculation of p_i is given by the following set of formulas:

$$p_i = \rho g \sum_{n=1}^{\infty} C_n \tanh m_n d \cos m_n z \sin kt = X_2 \sin kt$$

where

$$C_n = 2e^{-\frac{m_n d (m_n d \sin m_n L + \cos m_n d - 1)}{m_n d (\sin m_n d \cos m_n d + m_n d)}}$$

and

$$k^2 = -m_n g \tanh m_n d \quad (k = \frac{2\pi}{T})$$

Calculated with three terms ($n = 1, n = 2, n = 3$) in the case of a small wave period ($\frac{L}{d} = 0.5$) and in the case of a wave having the limit steepness, it gives a pressure distribution as shown in Figure 10.

It could be considered that the corresponding total force is uniformly distributed on a vertical $\frac{P_i}{\rho g L} = 0.10$. Hence,

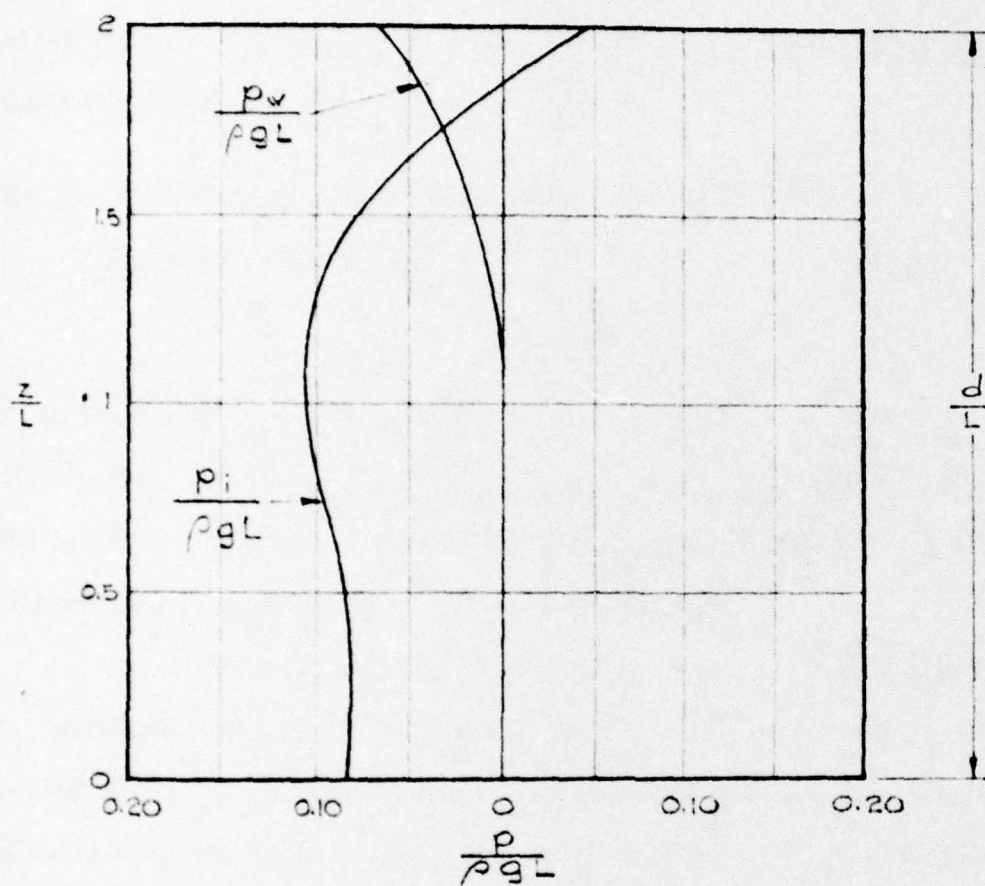
$$P_2 = 2\ell \int_0^d p_i dz = 2(0.10 \rho g L) d \ell$$

which gives for the W. B.: $2 \times 0.10 \times 62.4 \times 1.75 \times 3.5 \times 52 = 4,000$ lbs.,

and for the W. F. $2 \times 0.10 \times 62.4 \times 3.25 \times 6.5 \times 10 = 2,640$ lbs. In the

same condition P_1 is found to be 140 lbs.

MAXIMUM AMPLITUDE OF PRESSURE ON ONE SIDE OF A WAVE
PADDLE FOR $\frac{L}{d} = 0.5$ and $\frac{H}{L} = 0.14$



p_w : DUE TO WAVE MOTION.

$$p_w = \rho g \frac{H}{2} \frac{\cosh mz}{\cosh md} \cos kt$$

p_i : DUE TO INERTIA.

$$\begin{cases} p_i = \rho g \sum_{n=1}^{\infty} C_n \tanh m_n d \cos m_n z_n \sin kt \\ C_n = 2 \frac{m_n d \sin m_n d}{\sin m_n d \cosh m_n d + m_n d} \\ k^2 = -m_n g \tanh m_n d. \quad k = \frac{2\pi}{T} \end{cases}$$

CURVE EVALUATED FOR $n=1, n=2, n=3$.

FIGURE 10

FORCE DISTRIBUTION ON THE PADDLE DUE TO SHORT WAVES.

To know the maximum value for P_2 it would be necessary to calculate p_i for different values of L/d . p_i may begin to increase because of the increase of H_{\max} with T but also decreases quickly when the period increases because C_n decreases.

III-1.3) A Summary of Results Given by Theory

As a practical conclusion, the total force on the paddle is:

$$F = P_1 + P_2 = X_1 \cos kt + X_2 \sin kt.$$

X_2 is negligible for long waves and X_1 is 17,000 lbs. in the W. B. and 10,000 lbs. in the W. F. These forces are roughly linearly distributed on the paddle. P_1 is maximum when the paddle is vertical and acts in a direction which is opposite to the motion of the paddle.

X_2 is negligible for a short wave. ($X_2 = \frac{X_1}{14}$ when $L/d = 0.5$) X_1 becomes important for short waves. Calculated for the minimum wave period ($L/d = 0.5$) and the limit steepness, it is equal to 4,000 lbs. in the W. B. and 2,640 lbs. in the W. F. P_2 is zero when the paddle is vertical and maximum when the stroke is maximum. Its direction is away from the center vertical line. P_2 acts similar to an increase of inertia of the wave paddle itself.

Figure 11 summarizes these conclusions.

III-1.4) Practical Conclusion on the Wave Force

These forces given by the theory are not the maximum forces to be expected. The maximum forces are obtained for the smallest wave period and the maximum stroke. Then the wave breaks at the paddle and no theory

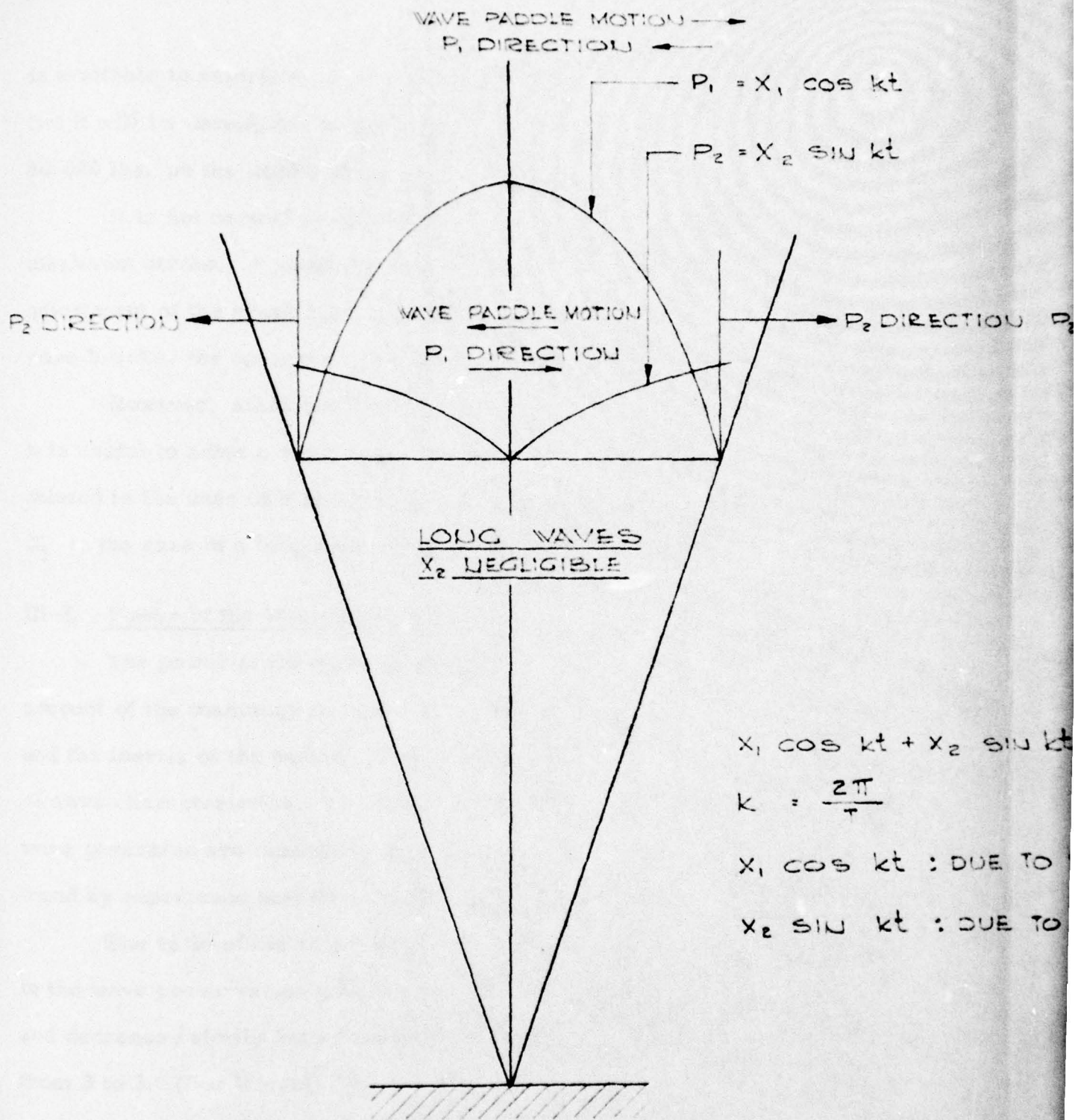
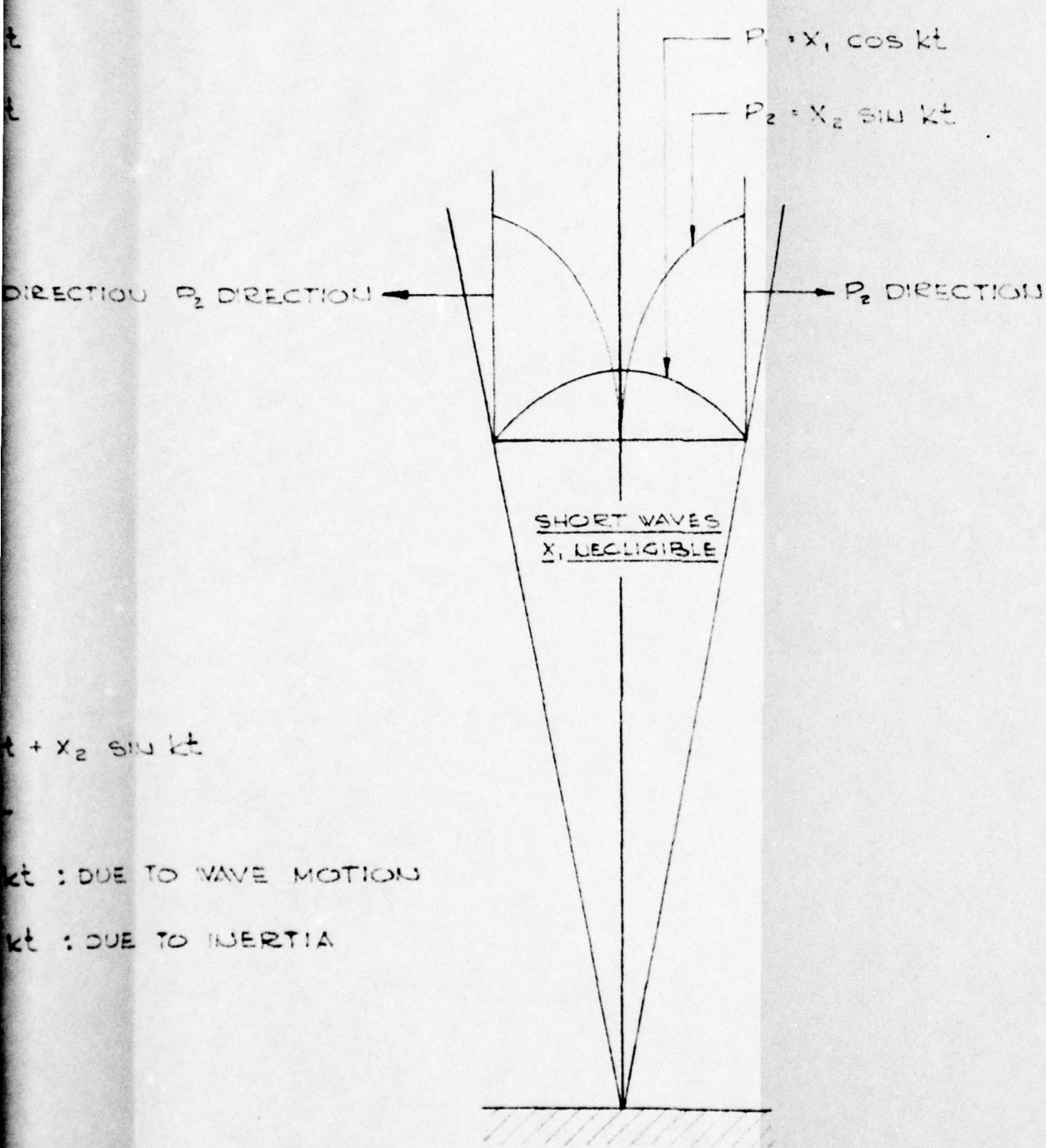


FIGURE II

FORCES ON WAVE PADDLE VS. T



$t + X_2 \sin kt$

kt : DUE TO WAVE MOTION

kt : DUE TO INERTIA

E VS. TIME

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is available to calculate the wave force. In that case X_1 is small (say $= \frac{X_2}{14}$). But it will be unrealistic to apply the theory to calculate X_2 . (It will give 60,000 lbs. on the paddle of the W. B.)

It is not normal to operate a wave paddle with minimum period and maximum stroke. A breaking wave at the paddle is possible only during the adjustment of the crank for a given wave height by trial and error. If the wave breaks, the operator stops the paddle at once and decreases the stroke.

However, since the abnormal operating condition may occur accidentally, it is useful to adopt a very large safety factor for X_2 as it has been calculated in the case of a short wave. A smaller safety factor is required for X_1 in the case of a long wave since the calculation is valid in that case.

III-2) Power of the Wave Generator

The power of the wave generator has to be calculated by taking account of the maximum power of the wave motion issued in both directions and the inertia of the paddle. The wave power is easily known as a function of wave characteristics. The total inertia and friction forces of the complex wave generator are difficult to calculate exactly. Hence they are usually found by experience based on the design and operation of other wave generators.

The ratio of the power required to move the wave generator to the wave power varies with the period. It is high for a small period and decreases slowly for a long period. This ratio varies within a range from 5 to 3. (See Wiegel⁽¹⁹⁾)

A multiplication factor is also required because of the fact that the power within a period varies considerably with time. The peak of power could be as high as 20 times the wave power in the case of short waves. However, this peak of power is provided by the fly wheel.

In fact, it must be expected that this ratio will be slightly greater for the W. B. than for the W. F. Indeed, the wave paddle 52 ft. long in the W. B. has relatively more inertia in order to keep its rigidity.

The maximum wave power is given by:

$$W. H. P. = \frac{2l}{550} \left[\frac{1}{2} \rho g \left(\frac{H}{2} \right)^2 \frac{C}{2} \left(1 + \frac{\frac{4\pi d}{L}}{\sinh \frac{4\pi d}{L}} \right) \right]$$

The coefficient 2 at the beginning is due to the fact that waves are issued in both directions. l is the length of the paddle. The following wave horsepowers have been calculated as an example:

	H (ft.)	T (sec.)	d (ft.)	c (ft./sec)	l	W. H. P.
W. F.	2.2	5	6.5	13.7	10	13.7
W. B.	1.1	4	4.5	10	52	16

IV. GENERATION OF IRREGULAR WAVES

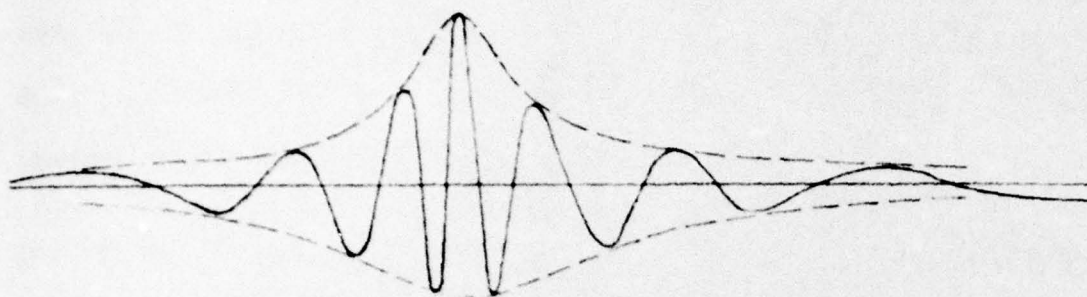
IV-1) Some Considerations on the Generation of Irregular Waves

In many studies it is of particular interest to reproduce a wave spectrum in the wave tanks. It is relatively easy to generate irregular waves but it is more difficult to generate a given wave spectrum. (See references 19, 20 and 21)

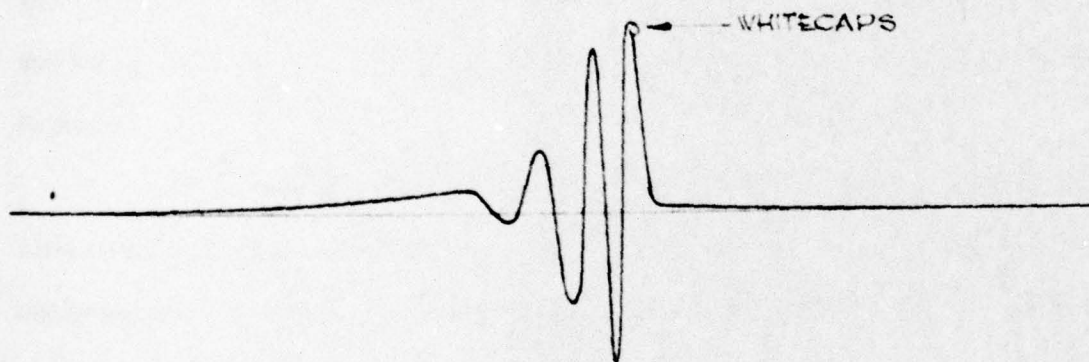
IV-1.1) The simplest way of generating irregular waves consists of changing the wave period. This change of wave period has a cycle which defines the period of the wave train. This variation could be adjusted by a cam or simply mechanically linearized. Then the variation of wave periods depends upon the mechanical system adopted for adjusting the wave period.

The change of wave period involves a change of L/d , i.e. a variation of the factor K (or K') according to the law presented by Figure 5. It is seen that changing the wave period automatically involves a change of wave height. But this variation, depending upon the relationship $K = f(L/d)$, cannot be controlled. Usually the wave train generated by such a variation gives a wave record near the paddle as presented by Figure 12.

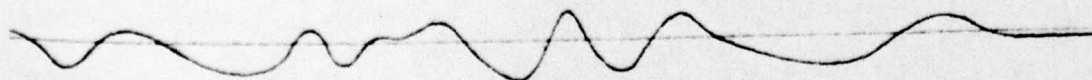
The longest wave corresponding to a small value of K (or K') has a small wave height. Conversely, the shortest wave has a large amplitude. But the wave train changes when it travels from the paddle to the testing area. The longest waves travel quickly and the shortest waves slowly. White caps appear due to the superimposition of a number n of irregular waves when the limit steepness is:



AT THE PADDLE



DEFORMATION WITH DISTANCE



IN THE TESTING AREA

FIGURE 12
GENERATION OF IRREGULAR WAVE BY CHANGING
THE WAVE PERIOD

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$$\sum \left(\frac{H}{L} \right)^2 > \frac{0.02}{n} \sum_{n=0}^n \text{Tanh}^2 \frac{2\pi d}{L}$$

For a given adjustment of the crank and a constant period of the wave train, these white caps always appear at the same place in the wave flume. Finally, in the operating area, the motion is very confused and irregular. The wave record is also different from one cross section to another.

IV-1.2) It has been seen that acting on the wave period also involves a variation of the wave height because of the relationships $H = K \lambda$ and $K = f(L/d)$. It is possible to change the wave height without changing the wave period. (Figure 13) Then the wave train travels without deformation and the wave record is the same at any cross section, with only a difference in phase.

This change in wave height could be carried out by changing the eccentricity at the crank with the period of the wave train. But this is mechanically more difficult to realize because it involves tremendous forces on the crank parts.

Another method consists of plunging a wave absorber with the period of the wave train in the middle of the wave filter. This wave absorber may also be a simple vertical barrier which partly reflects the wave motion. A variable part of wave energy is transmitted under this movable barrier and the bottom of the flume.

IV-1.3) The most refined method consists of generating a wave spectrum by a servo-motor. Then it is theoretically possible to reproduce any wave train. But it is often difficult to calculate the cam which commands the

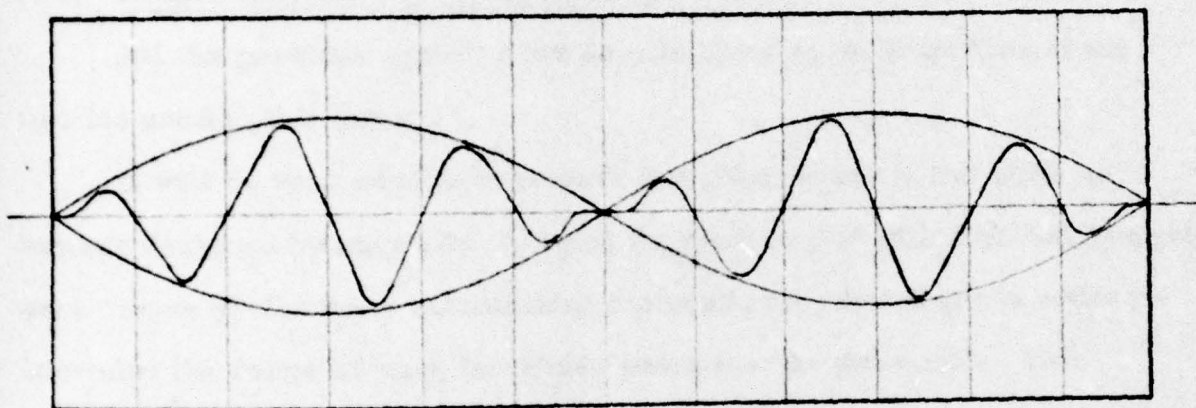


FIGURE 13
WAVE TRAINS OBTAINED BY CHANGING THE ECCENTRICITY
AT THE CRAUK.

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servo-motor. This cam must be designed by taking into account the response of the servo-motor and also the wave characteristics as a function of the motion of the paddle. Both vary with period. It is theoretically possible, but the servo-motor must be very powerful to have a good linear response.

IV-2) Adaptability for Producing Irregular Waves

All the previous systems can be calculated by an adaptation of the theories used in this report.

It will be very easy to reproduce irregular waves in the tanks as they are designed by manually changing the wave period with a period of wave train. Some preliminary calculations can easily be carried out in order to determine the range of wave amplitude which may be generated. This could be done by using the relationship $K = f(L/d)$. Later on a device could be adapted to reproduce cycles of variation of wave period.

It will also be possible to change the wave amplitude without changing the wave period by a slight modification: an intermediate lever can be inserted in the W. F. between the crank and the paddle. The primary purpose of this intermediate lever is to decrease the length of the required crank and the maximum stroke of the paddle. This intermediate lever can easily be changed or modified in order that the length of one of its arms will vary with the period of wave train. By changing this length, the stroke is automatically changed and consequently, the wave height is changed.

Insertion of a wave absorber in the middle of the wave filter may also be considered.

V. OPERATION OF WAVE GENERATOR

V-1. Wave Paddle Drive and Amplitude Adjustment

The power requirement for the W. B. has been set nominally at 60 H. P. as previously discussed in Section II, III-2. Since this power must be applied to the paddle drive to provide as nearly sinusoidal paddle motion as possible, the driving mechanism must be noncompliant, whereby there is no phase shift between power generator and the wave paddle. This should also be accomplished without excessive weight of linkages or other reciprocating or oscillating mechanisms to minimize inertial loads.

The most practical arrangement is shown by Figure 14 for the W. B., and Figure 15 for the W. F., whereby an offset crank arm and sufficiently long connecting rod is attached to the paddle at a point above the maximum water level. This geometry more nearly simulates an exact sinusoidal paddle motion than the conventional arrangement in which the connecting rod is horizontal at maximum stroke.

The amplitude of the wave paddle is varied by adjusting the length of the crank arm. This is accomplished manually on the W. B. by a handwheel and worm gear arrangement as shown on the drawing. A vernier scale is provided for close adjustment, especially for small stroke settings. Although a sturdy square thread is used to position the connecting rod bearing, the major driving forces are transmitted by V guides which may be adjusted for clearance or locked to prevent any movement during operation. An automatically adjusted counterweight is provided to balance the connecting rod end and bearing.

The adjustable crank arm for the W. F. may be provided with

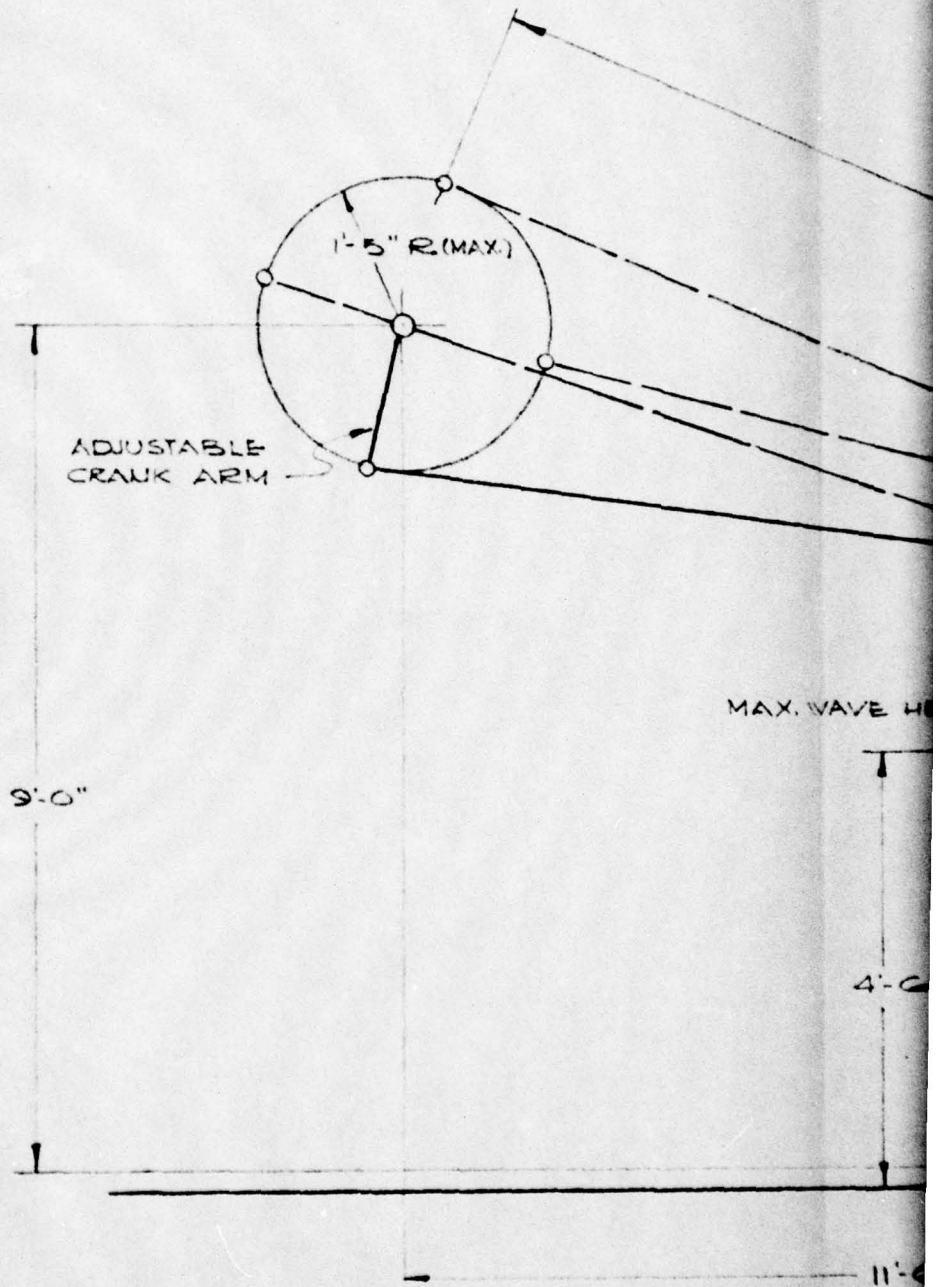


FIGURE 14

WAVE BASIN PADDLE DRIVE ARM

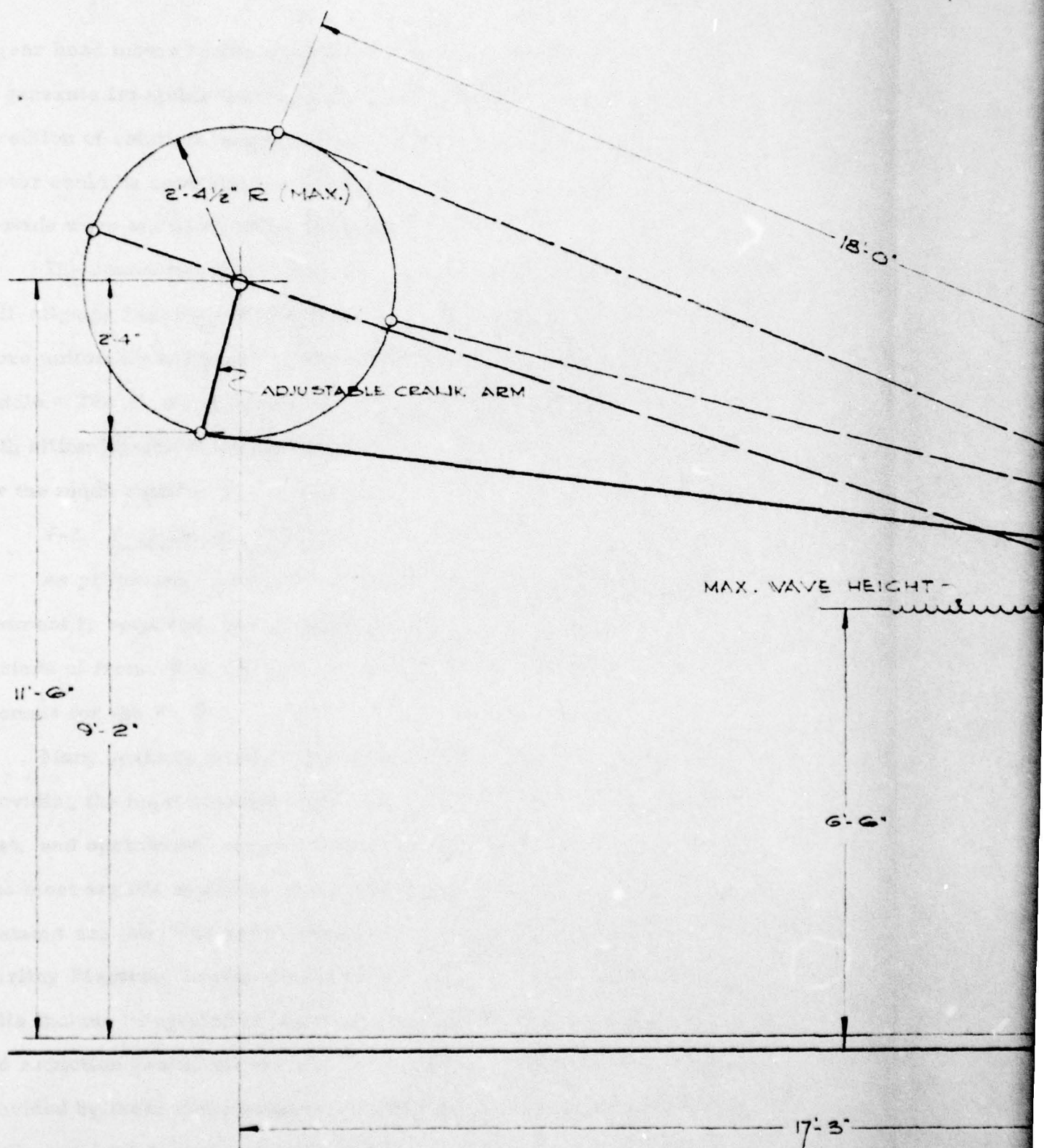
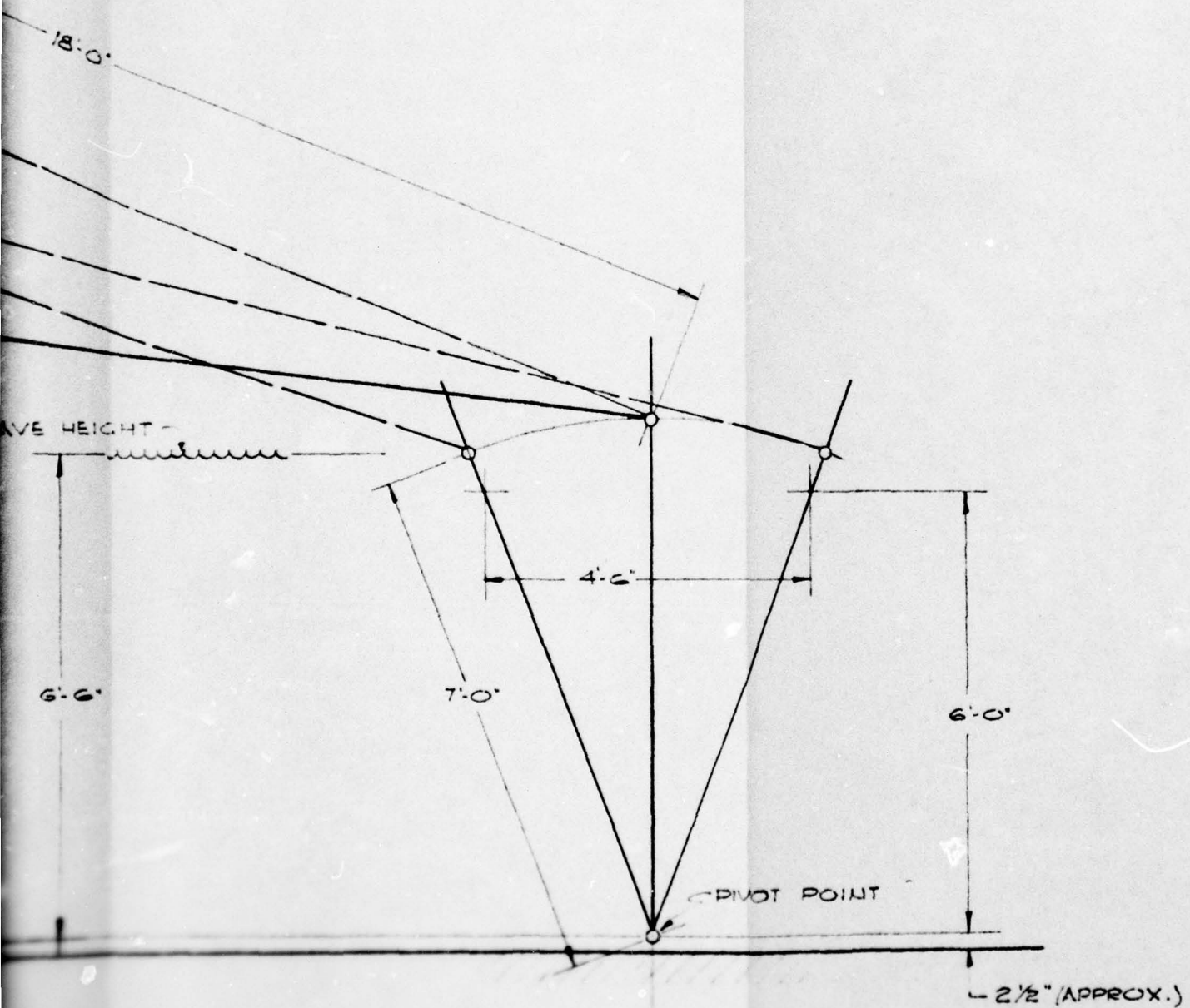


FIGURE 15

WAVE FLUME PADDLE DRIVE ARRANGEMENT



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a gear head motor to remotely control the amplitude during operation to generate irregular waves as discussed in Section II, IV-1. The direction of rotation, and if necessary, the speed of rotation of this motor could be controlled by a suitable programming system to provide wave trains or other wave spectrum.

The connecting rod is attached to the W. B. paddle through sealed self-aligning bearings at two points, both to transmit the driving force more uniformly and also to reduce possible deflection in the long paddle. The W. B. is also provided with a forked connecting rod with stiffening struts to prevent bending. This feature is not necessary for the much smaller W. F. paddle.

V-2. Variable Speed Drive

As previously specified, a continuous speed or wave period adjustment is required over at least a range of 1 to 6 to provide wave periods of from .5 to 3.0 seconds for the W. B. and from .7 to 4.2 seconds for the W. F.

Many systems were considered to determine the best method for providing the most constant angular velocity, compactness, lowest cost, and operational safety for the many possible operating conditions. The most readily available stock electric motor driven variable speed systems are the "Varidrive" type units as made by U. S. Motors, Sterling Electric, Reeves-Reliance Electric, and Link-Belt. These units include integrated or close coupled motor, variable speed changer, and reduction gears, all mounted on one base. From available ratings provided by these companies, a standard stock size most nearly adapted to the required service conditions was a 60/30 H. P. Reeves-Reliance system using a two-speed motor and 4 to 1 speed changer with an

over-all wave period variation of approximately 1 to 8.

It was noted that all of these continuous speed change units were torque limited; that is, at the upper speed range constant power could be transmitted, but at the low speed range only a limiting or constant torque was available. This limiting torque at the very low speed required for the longest wave period in some cases reduced the available output power to less than one-half the nominal motor rating. Also, for the speed range required, it was not possible to obtain an over-size variable speed unit to deliver maximum full motor power of 60 H. P. at the lowest speed necessary for the longest wave period.

A more detailed analysis was made to determine the relation between available and required torque for the varied range of wave periods and wave heights under consideration. The results as shown in Figures 16 and 17 indicate a standard variable speed system available from Reeves Division of the Reliance Electric & Engineering Co. would meet most of the operating conditions.

If greater torque is considered necessary for the longer wave periods, the available torque from the low speed range can be approximately doubled by using a two-speed gear reducer with a speed changer using a single-speed 60 H. P. motor. This would appear to be a more satisfactory arrangement. However, two speed gear reducers (with manual gear shift) are not available as stock items for the speed ratio required (over 40 to 1), and the resulting cost of specially designed units may appreciably increase the price of the drive system. Two speed gears of this type are made to specifications by Wester Gear Corp. and probably others.

Another alternate to the two-speed gear reducer is a chain drive system with a two-speed sprocket and shift mechanism. This, however,

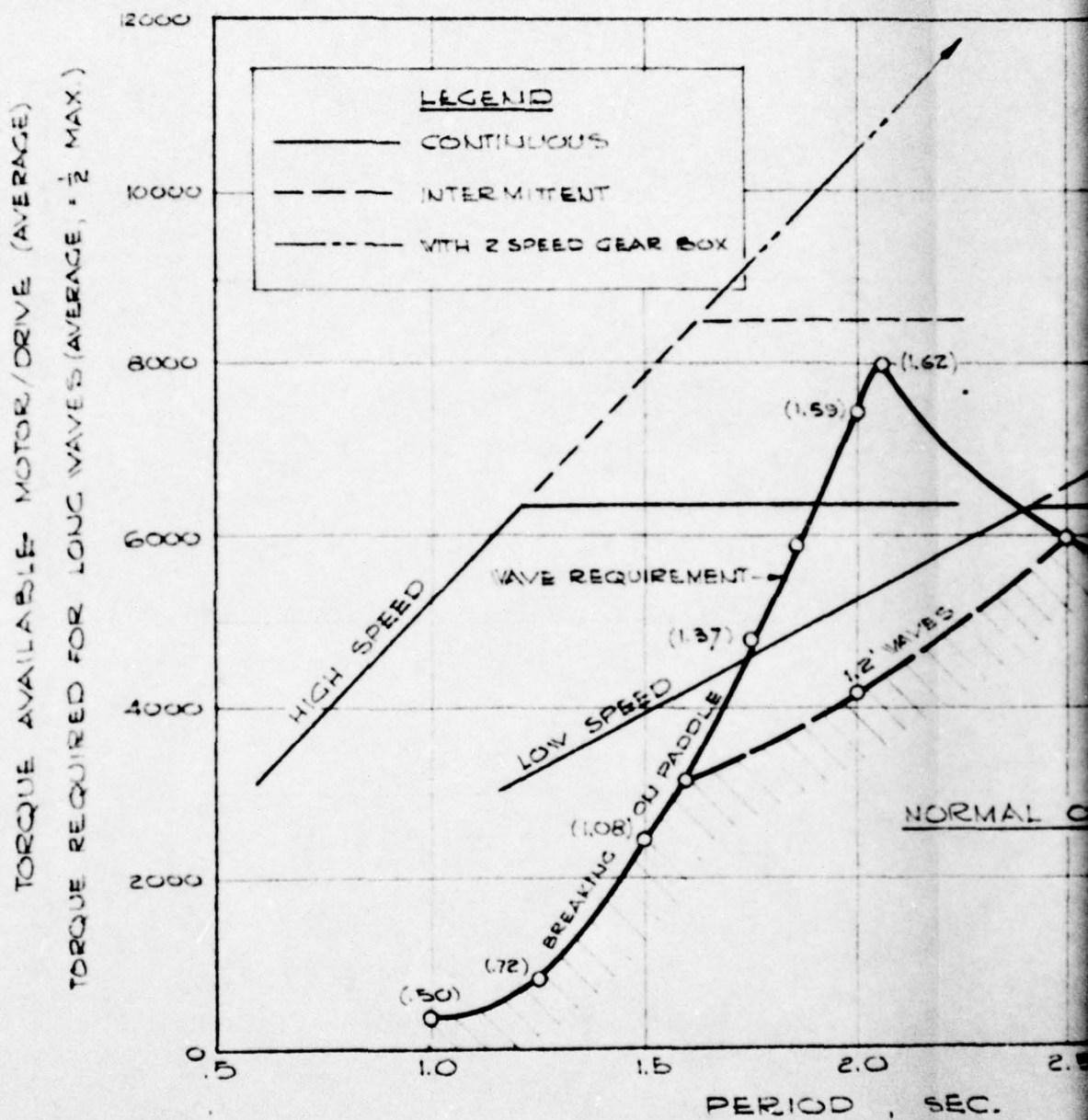
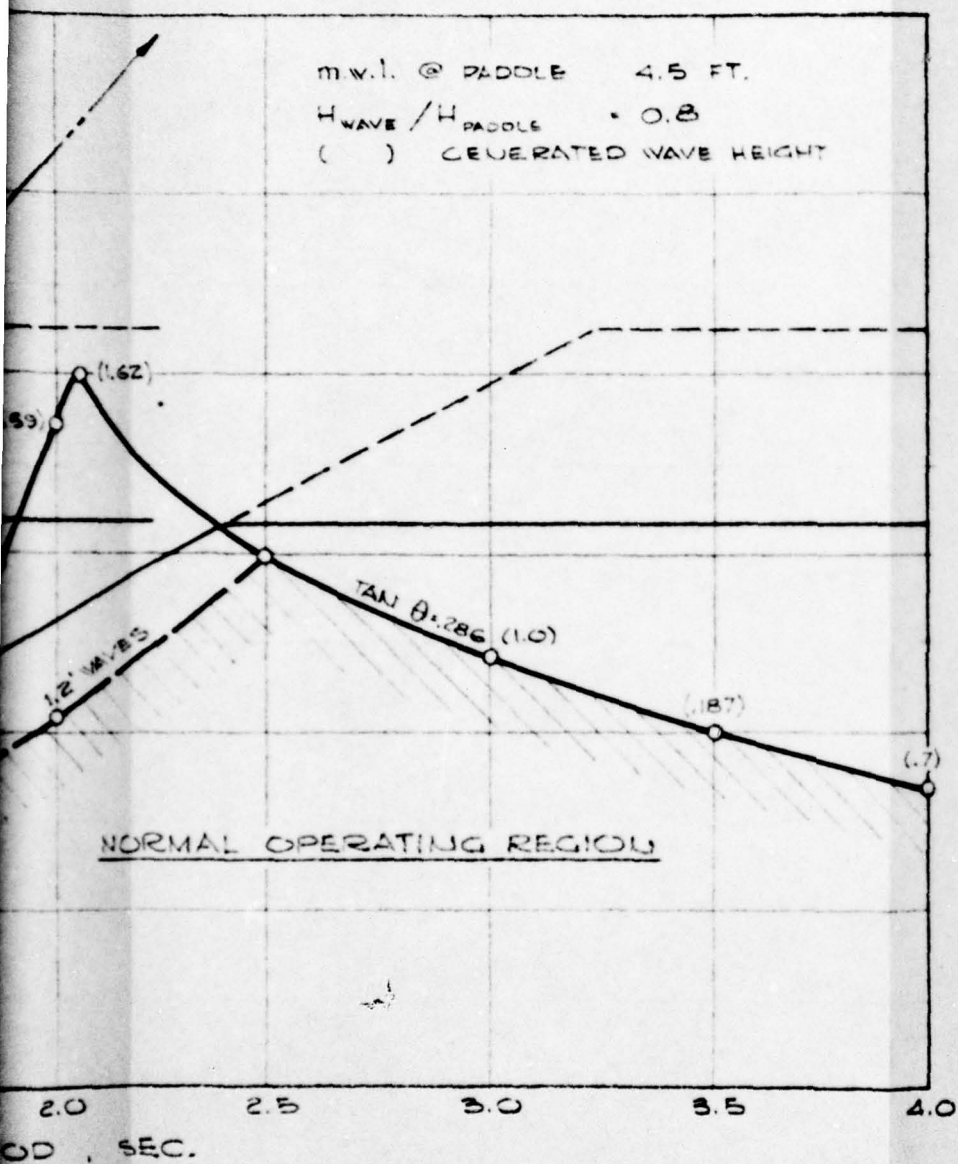


FIGURE 16 .

MOTOR / DRIVE CAPABILITIES, WA
WAVE BASIN



ABILITIES, WAVE REQUIREMENTS VS. PERIOD

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TORQUE AVAILABLE MOTOR/DRIVE (AVERAGE)
 TORQUE REQUIRED FOR LONG WAVES (AVERAGE, $\approx \frac{1}{2}$ MAX.)

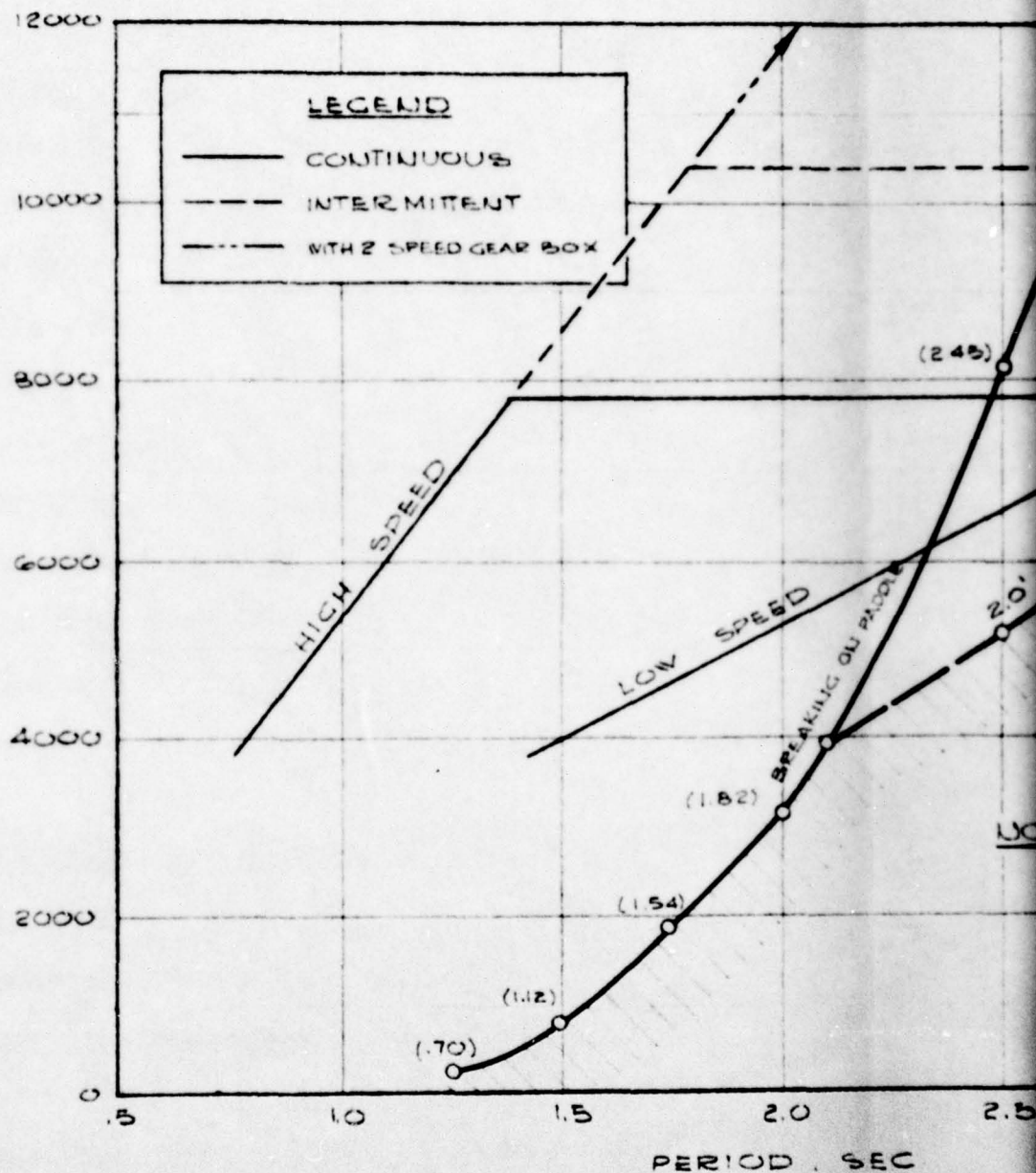
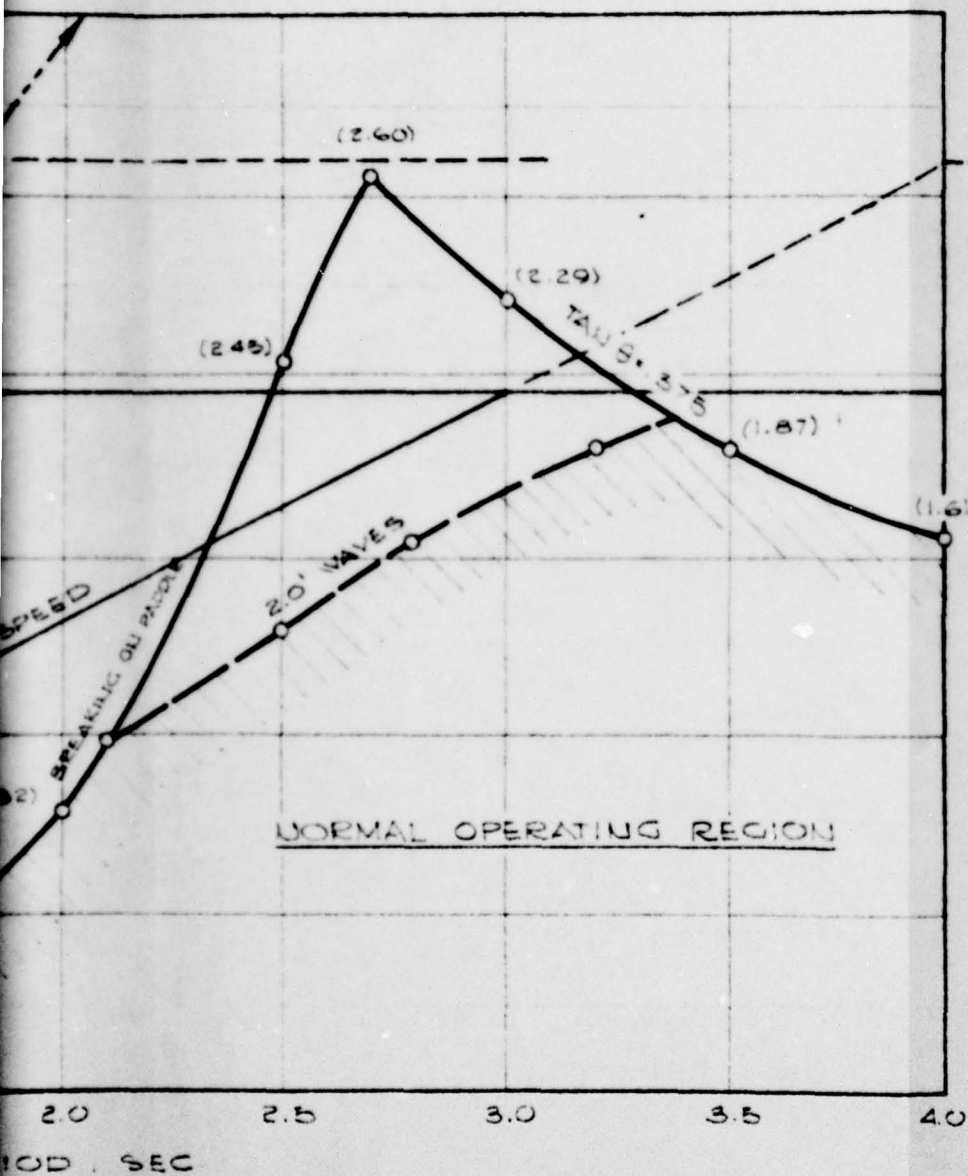


FIGURE 17

MOTOR / DRIVE CAPABILITIES, WAVE REQ
WAVE FLUME



M.M.L. @ PADDLE 6.5 FT.

$H_{\text{WAVE}} / H_{\text{PADDLE}} = 0.8$

() GENERATED WAVE HEIGHT

WAVE REQUIREMENTS VS. PERIOD

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requires a jack shaft, extra bearings, chain drive and guard and clutch couplings, but would provide the two-speed reduction at somewhat lower cost than the two-speed gear reducer.

V-3. Wave Paddle Drive Controls

The motor required for the variable speed wave paddle drive should be provided with a two-speed magnetic non-reversing starter with thermal overload relays for a 60/30 H. P. constant torque motor, 60 cycle, 3 phase, 440 volt. This type starter will protect the motor for any motor overload condition during operation.

However, conditions could occur where the motor and variable speed unit could be greatly overloaded, especially during starting when a cold motor could generate over 200% torque. Under these conditions, damage may occur to the variable speed unit. To prevent this possibility, an interlocking control is necessary to insure that the speed control is always set at minimum speed during the start of any test.

This is provided on the Reeves-Reliance Electric system by a time delay relay which energizes a remote control motor to return the Reeves Drive to low position before the motor is stopped. This will insure starting the drive at minimum speed during subsequent operation.

The operator controls are supplied with 60 cycle, single phase, 120 volt current, and provide "low" and "high speed run" push buttons and a "stop" push button which is interconnected with the time delay relay for return to low speed.

An electric remote speed control is provided with "fast" and "slow" push buttons to change drive speed. A tachometer generator

mounted on the Reeves Drive provides a voltage signal to speed indicating meter which may be calibrated directly for wave period or drive speed.

A panel board would also be provided for mounting all motor controls, remote setting buttons, and speed indicator located at selected operator station.

CONCLUSION

It would be desirable to use the available nomographs which have been calculated for this report to verify the presented theories. This will permit, in particular, the determination of the coefficient of efficiency η of the wave paddles, a datum which can be of great interest in other eventual installations.

It will also be desirable to verify the breaking conditions at the paddle which have been proposed in this report.

Finally, a preliminary test will be required in order to determine, experimentally, the wave filter coefficient which permits calculation of its damping effect.

The theoretical work which has been carried out and presented in this report can also be continued. In particular, it will be of great interest to investigate further the forces on a paddle both for breaking and non-breaking waves. These theories could be substantiated or modified after a series of measurements.

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PROJECT PERSONNEL

Technical Requirements provided by:

Mr. Robert Y. Hudson
Mr. John G. Housley

U.S. Army Engineers
Waterways Experiment
Station, Vicksburg, Miss.

Mr. Charles E. Lee
Mr. G. D. Smith

Office of Chief Engineers
U.S. Army Corps of Engineers
Washington 25, D.C.

Project Supervisor:

Dr. Charles L. Bretschneider
Washington Office
National Engineering Science Co.

Hydraulic Design:

Dr. Bernard Le Mehaute
Washington Office
National Engineering Science Co.

Structural Design:

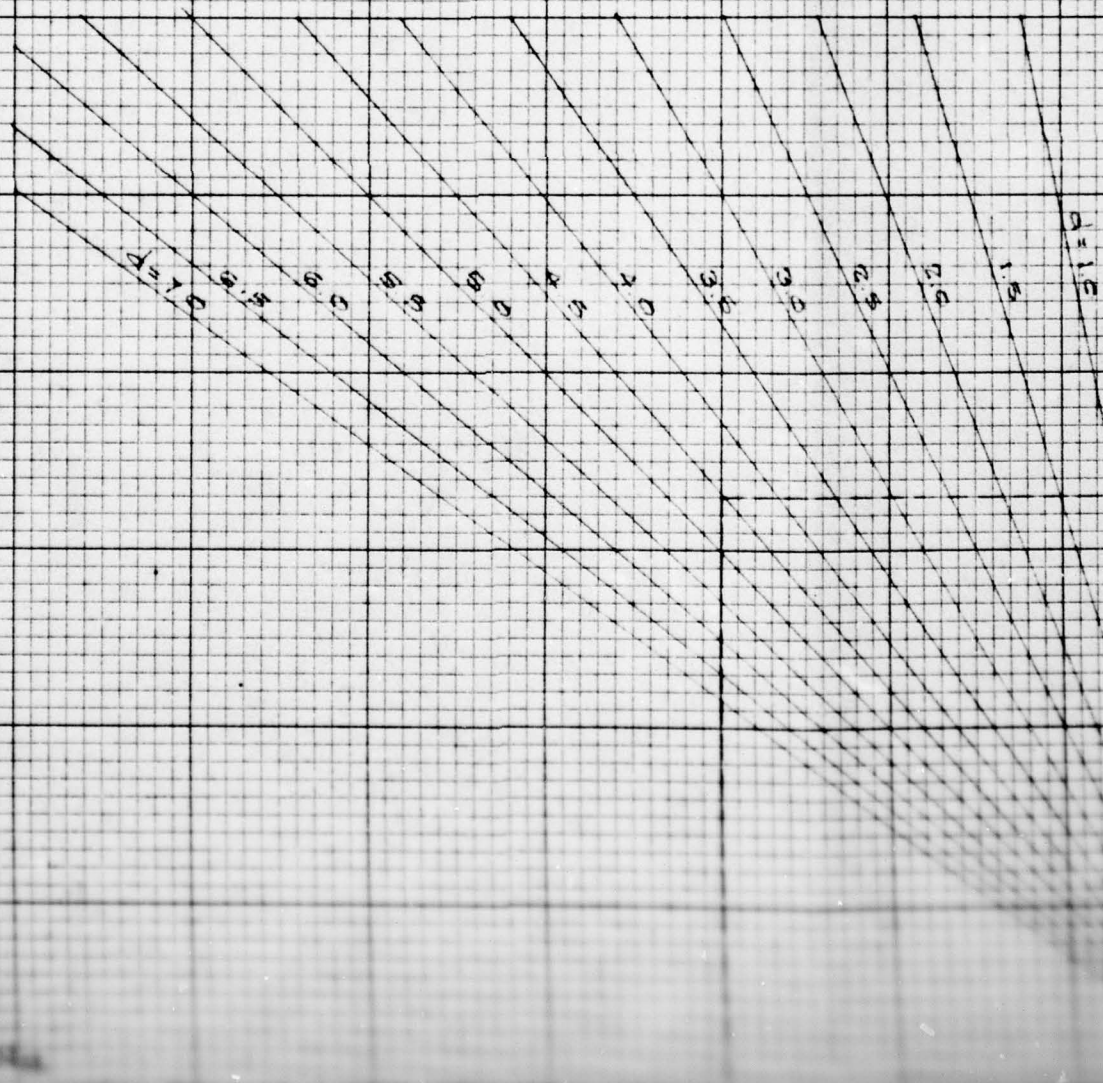
Mr. Raymond C. Hurt
Mr. William A. Sandberg
Mr. Arlen W. Bell
Pasadena Office
National Engineering Science Co.

Mechanical Design:

Mr. H. Edmund Karig
Pasadena Office
National Engineering Science Co.

Drawings:

Mr. Tak Umehara
Mr. William Rogers
Mr. Guido Zemgals



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DESIGN OF WAVE TANKS, (U)
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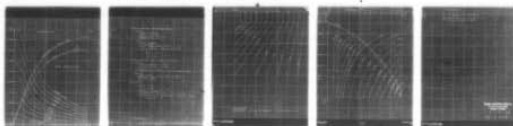
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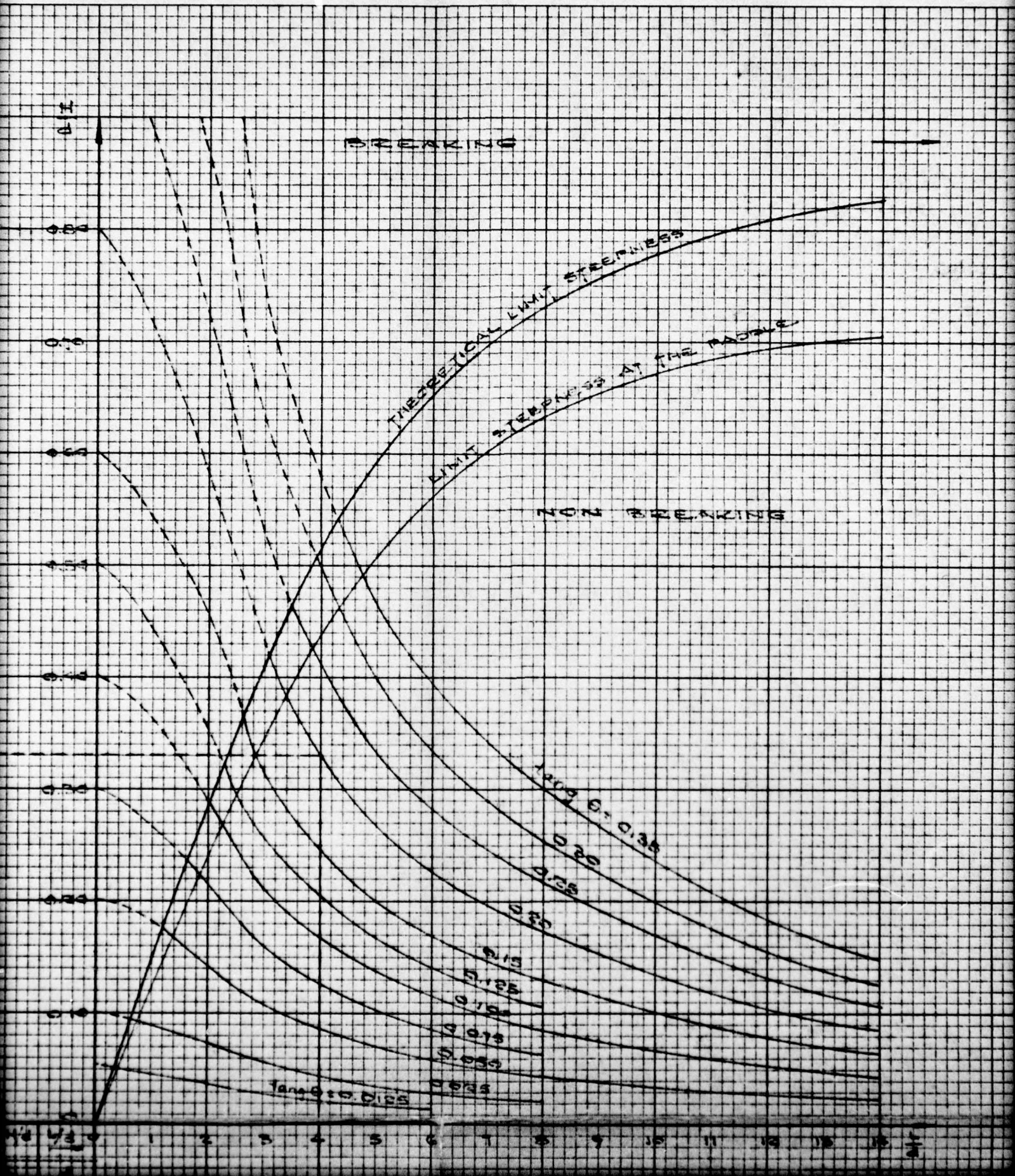
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CALCULATION OF WAVE HEIGHT

INPUT:

DEPTH $d = 4.5$ ft
 PERIOD $T = 2.5$ sec
 STROKE: $\text{tang } \theta = 0.20$
 EFFICIENCY $\eta = 0.71$

OUTPUT:

DO NOT BREAK
 H THEORETICAL: $H_{th} = 1.49$ ft
 H REAL (EFFICIENCY $\eta = 0.71$): $H = 1.06$ ft
 H AFTER FILTER ... (KT = 0.5): $H = 0.74$ ft

ALSO:

$T^2/d = 0.7$
 $L/d = 4$ $L = 18$ ft
 $H/d = 0.33$

CALCULATION OF A WAVE PADBLE

INPUT:

DEPTH (MINIMUM): $d = 1.5$ ft
 PERIOD (MAXIMUM): $T = 2.5$ sec
 EFFICIENCY $\eta = 0.8$
 WAVE HEIGHT (MAXIMUM) BEFORE FILTER: $H = 1.49$ ft

OUTPUT:

WAVE HEIGHT (THEORETICAL) $H = 1.49$ ft
 STROKE: $\text{tang } \theta$ (MINIMUM) $= 0.30$
 DO NOT BREAK

NOTATION

DEPTH d ft
 PERIOD T sec
 WAVE LENGTH L ft $m = 2\pi/L$
 STROKE: $\text{tang } \theta$
 EFFICIENCY OF THE PADBLE η (KL 0.8)
 EFFECT OF FILTER, CONVERGENT CHANGE OF DEPTH:
 X (3) OR Y (1)

$$H = 2K' d \text{ tang } \theta$$

$$K' = \frac{1}{m} \left(\sin m d \cos m d + m d \right)$$

BREAKING CONDITIONS AT THE PADBLE

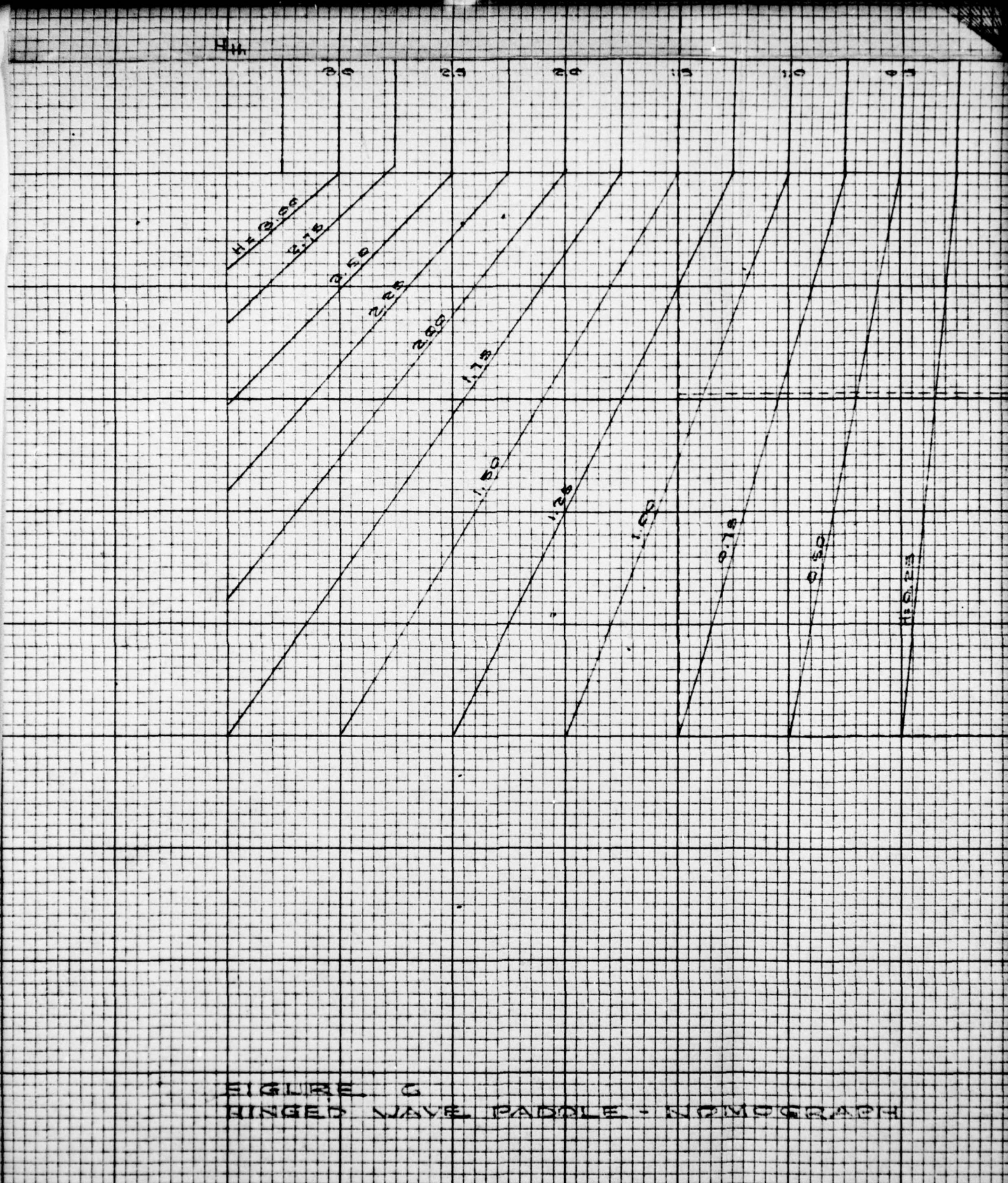
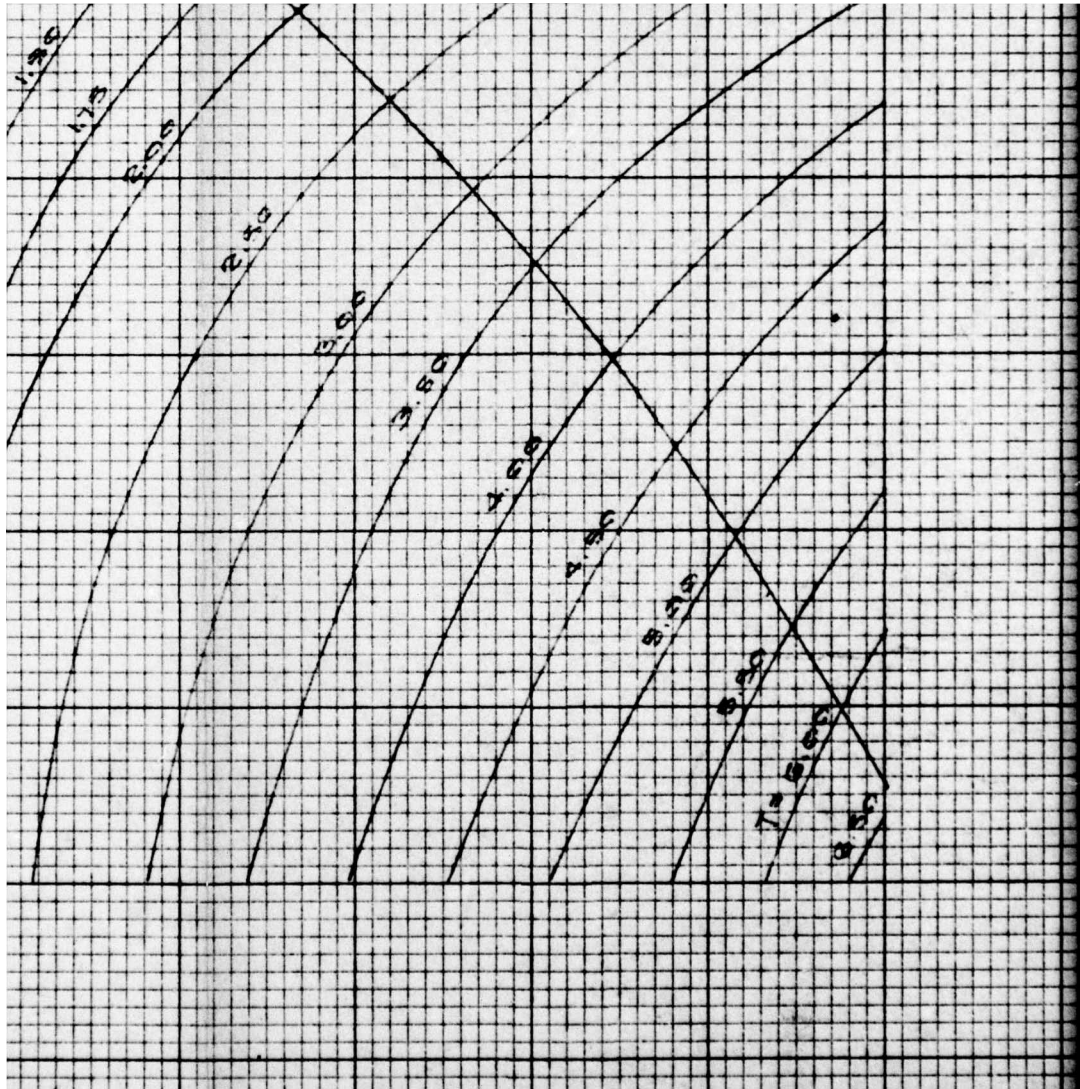


FIGURE 6
HINGED WAVE PADDLE - NOMOGRAPH



4 = 2K² Tang @

K = m₀ (sin² m₀ cosh m₀ + m₀²)

BRACKING SECTION AT THE TAIL

$$T/D = 0.18 \frac{L}{L} \tanh \frac{ST_0}{L}$$

NATIONAL ENGINEERING SCIENCE CO.
701 SOUTH FAIR OAKS AVENUE
PASADENA, CALIFORNIA

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